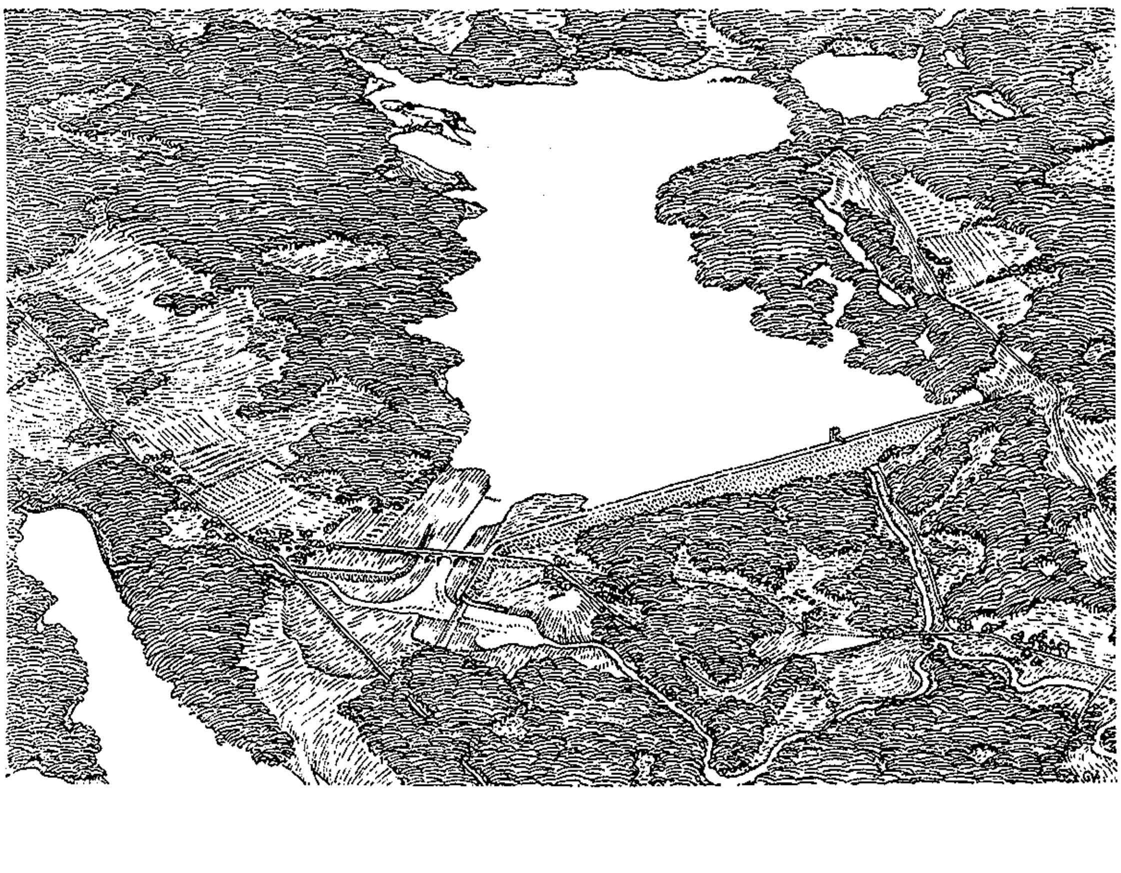
MERRIMACK VALLEY FLOOD CONTROL

DEFINITE PROJECT REPORT

on

BEARDS BROOK RESERVOIR

HILLSBORO—NEW HAMPSHIRE



NOVEMBER—1945

CORPS OF ENGINEERS—U. S. ARMY
U. S. ENGINEER OFFICE—BOSTON—MASSACHUSETTS

DEFINITE PROJECT REPORT

ON

BEARDS BROOK RESERVOIR
NEW HAMPSHIRE

PREPARED IN THE U.S. ENGINEER OFFICE,

BOSTON, MASSACHUSETTS DATED NOV., 1945

APPROVED BY THE CHIEF OF ENGINEERS 1945

E.D. FILE ______________________________

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DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

MERRIMACK VALLEY FLOOD CONTROL

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PERTINENT DATA

DEFINITE PROJECT REPORT
BEARDS BROOK RESERVOIR

SECTION A - PERTINENT DATA

A summary of the pertinent data and governing criteria for the selection, design, construction, operation and cost of the Beards Brook Reservoir, Merrimack River Basin, New Hampshire, as described in this report is presented in the following table.

1. Project Location .... North Branch and Beards Brook, tributaries of the Contoocook River; approximately 2 miles west of the village of Hillsboro and 8 miles north of the village of Bennington, N.H., approximately at river mile 136 above the mouth of the Merrimack River.

2. Reservoir Data

Drainage Area:	
North Branch, Contoocook River	64 sq. miles
Beards Brook	56 sq. miles
Total Drainage Area	120 sq. miles
Flood control storage capacity	35,000 acre-feet
Inches of storage capacity on drainage area...	5.5 inches
Reservoir area at elevation of spillway crest:	
Heavily wooded land (69%)	556 acres
Sparse timber land (8%)	67 acres
Cleared and cultivated land (15%)	119 acres
Water and swamp land (8%)	68 acres
Total reservoir area	810 acres
Length of reservoir	2 miles
Maximum wave fetch	2 miles

3. Stream Flow Data

Average flow, North Branch, Contoocook River..	115 c.f.s.
Average flow, Beards Brook (estimated)........	100 c.f.s.
Average annual flood, North Branch, Contoocook River	1,650 c.f.s.
Average annual flood, Beards Brook (estimated)	1,800 c.f.s.
Flood of 19 March 1936, North Branch, Contoocook River	5,400 c.f.s.
Flood of 19 March 1936, Beards Brook(estimated)	6,300 c.f.s.
Flood of 21 September 1938, North Branch, Contoocook River	4,450 c.f.s.
Flood of 21 September 1938, Beards Brook (estimated)	6,000 c.f.s.

3. Stream Flow Data (Cont'd.)

Spillway design flood-inflow to reservoir:	
North Branch, Contoocook River	23,900 c.f.s.
Boards Brook	57,000 c.f.s.
Total design flood	66,500 c.f.s.
Spillway design flood-outflow	62,500 c.f.s.
Spillway design capacity (allowing for failure of Jackman Dam)	79,300 c.f.s.

4. Dam

Type	rolled earth
Total length of dam	4,900 feet
Maximum height of earth embankment	108 feet
Top width of earth embankment	25 feet
Spillway crest elevation M.S.L.	705 feet
Spillway surcharge (Spillway Design Flood)	11 feet
Spillway surcharge (allowing for failure of Jackman Dam)	12.9 feet
Canal head loss	1 foot
Allowance for failure of Jackman Reservoir	1 foot
Freeboard	5 feet
Top of dam elevation	723 feet MSL
Power development	none

5. Outlets

Location (a) Embankment	single gated conduit
(b) Spillway	single ungated outlet
Embankment conduit:	
Size of conduit	64 sq. ft.
Number of service gates	3
Type of gates	Hydraulically operated sluice gates
Size of gate openings	2 gates - 3'-0 x 7'-0
....	1 gate - 4'-0 x 7'-0
Elevation of gate sills	617 feet MSL
Maximum discharge capacity (W.S. El. 705)...	3,360 c.f.s.
Spillway outlet:	
Size of outlet	4'-0" x 6'-0"
Number of service gates	none
Elevation of outlet invert at intake	680 feet MSL
Maximum discharge capacity (W.S. El. 705)...	860 c.f.s.
Downstream channel capacity (estimated)	2,500 to 3,000 c.f.s.
Time of emptying 80% of capacity	7 days

6. Spillway

Type of spillway	concrete gravity ogee
Crest elevation	705 feet MSL
Length	450 feet
Spillway design discharge	80,000 c.f.s.
Maximum reservoir elevation:	
(Spillway design flood)	716 feet MSL
(Allowing for failure of Jackman Dam)	717.9 feet MSL
Allowance for failure of Jackman Reservoir	2 feet
Design head	13 feet

7. Foundations

Dam, general	Impervious glacial till
Dam, core	Impervious glacial till
Dike	Impervious glacial till
Outlet works	Impervious glacial till
Diversion channel weir	Impervious glacial till
Spillway	Bed rock
Spillway retaining walls	Bed rock

8. Quantities

Embankment:	
Impervious fill	290,000 c.y.
Pervious fill	460,000 c.y.
Random fill	280,000 c.y.
Semi-compacted fill	40,000 c.y.
Special and processed gravel	116,000 c.y.
Rock slope protection	185,000 c.y.
Total Embankment	1,371,000 c.y.
Excavation:	
Earth	380,000 c.y.
Rock ledge	75,000 c.y.
Boulders	25,000 c.y.
Total Excavation	480,000 c.y.
Concrete:	
Outlet works	7,000 c.y.
Spillway	5,000 c.y.
Spillway walls	2,800 c.y.
Diversion canal spillway and walls	1,800 c.y.
Miscellaneous items	5,200 c.y.
Total Concrete	21,800 c.y.

9. Estimated Cost (Including contingencies, overhead, etc.)

Reservoir Clearing	$ 16,000
Reservoir Costs	244,000
Construction Costs	2,920,000
Total Cost	$ 3,180,000

Cost per acre-foot of total storage = $ 90.86

SECTION B

SYLLABUS

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

SECTION B - SYLLABUS

The Beards Brook Reservoir is proposed as a part of the comprehensive plan for flood control of the Merrimack River Basin, authorized by the Flood Control Act approved 22 June 1936 and amended by the Flood Control Act approved 28 June 1938.

The reservoir site is located in the town of Hillsboro, Hillsboro County, New Hampshire, in the Beards Brook drainage basin, approximately one and one-half miles upstream from the confluence of the North Branch and the Contoocook River. The dam and spillway are laid out to impound flood waters of the North Branch and Beards Brook, and are designed to provide flood protection for the town of Hillsboro and to reduce flood stages on the lower reaches of the Contoocook and Merrimack Rivers. The proposed dam consists of an earth-filled embankment section, approximately 4,400 feet long with a maximum height of 108 feet, constructed across the Beards Brook Valley. The proposed spillway is a concrete gravity ogee section with a crest length of 450 feet and is founded on bedrock across the North Branch Valley. The outlet works, consisting of an intake structure, three (3) hydraulically operated sluice gates, a single conduit through the embankment and a stilling basin, are located approximately in the existing stream bed of Beards Brook. An ungated outlet is provided in the spillway section for discharging the normal overflow from the power pool of the Jackman Reservoir which is located approximately 2,500 feet upstream on the North Branch. A short channel, located immediately upstream from the embankment section and spillway is provided to divert flood waters from the North Branch into the reservoir, and to act as a spillway approach channel from the reservoir for spillway floods. The estimated cost of the project is $3,180,000, based on a 3-yesr construction period. Local financial cooperation is not required for this project. Authorization for preparation of this project report is contained in letter from the Chief of Engineers, Washington, D. C., to the Division Engineer, New England Division, dated 18 December 1943, subject: "Definite Project Reports for Bennington and Beards Brook Reservoirs", (OCE File No. CE 821.2 Hopkinton-Everett Dam)).

SECTION C

TEXT OF REPORT

War Department
United States Engineer Office
Boston, Massachusetts

MERRIMACK RIVER BASIN FLOOD CONTROL PROJECT

DEFINITE PROJECT REPORT
BEARDS BROOK RESERVOIR
HILLSBORO, N.H.

SECTION C - TEXT OF REPORT
November 1945

1. Project Authorization.- The Beards Brook Reservoir for flood control in the Contoocook River Basin, New Hampshire, as described herein is proposed as an element of the comprehensive plan for flood control reservoirs and related flood control works for the Merrimack River Basin authorized by the following portions of the Flood Control Acts of 1936 and 1938:

a. Flood Control Act, approved 22 June 1936 (Public No. 738 - 74th Congress):

"FLOOD CONTROL ACT OF 1936

"Sec. 5. That pursuant to the policy outlined in Sections 1 and 3, the following works of improvements, for the benefit of navigation and the control of destructive flood waters and other purposes, are hereby adopted and authorized to be prosecuted, in order of their emergency as may be designated by the President, under the direction of the Secretary of War and supervision of the Chief of Engineers in accordance with the plans in the respective reports and records hereinafter designated: Provided, That penstocks or other similar facilities, adapted to possible future use in the development of adequate electric power may be installed in any dam herein authorized when approved by the Secretary of War upon the recommendation of the Chief of Engineers.

* *

"MERRIMACK RIVER, NEW HAMPSHIRE AND MASSACHUSETTS

"Construction of a system of flood control reservoirs in the Merrimack River Basin for the reduction of flood heights in the Merrimack Valley generally; estimated construction cost, $7,725,000; estimated cost of lands and damages, $3,500,000."

b. Flood Control Act, approved 28 June 1938 (Public No. 761 - 75th Congress, 3rd Session):

"Sec. 4. That the following works of improvement for the benefit of navigation and the control of destructive flood waters and other purposes are hereby adopted and authorized to be prosecuted under the direction of the Secretary of War and supervision of the Chief of Engineers in accordance with the plans in the respective reports hereinafter designated: Provided, That penstocks or other similar facilities adapted to possible future use in the development of hydroelectric power shall be installed in any dam herein authorized when approved by the Secretary of War upon the recommendation of the Chief of Engineers and of the Federal Power Commission.

"MERRIMACK RIVER BASIN

"The general comprehensive plan for flood control and other purposes, as approved by the Chief of Engineers pursuant to preliminary examinations and surveys authorized by the Act of June 22, 1936, is approved and the project for flood control in the Merrimack River Basin, as authorized by the Flood Control Act approved June 22, 1936, is modified to provide, in addition to the construction of a system of flood control reservoirs, related flood control works which may be found justified by the Chief of Engineers."

The Beards Brook Reservoir is selected in accordance with the recommendation of the Board of Engineers for Rivers and Harbors which was concurred in by the Chief of Engineers in the 10th Indorsement, dated 10 December 1943 on letter from the Chief of Engineers to the Resident Member, Board of Engineers for Rivers and Harbors, dated 6 December 1941, subject, "Reservoir Plans for the Contoocook Basin, New Hampshire", File No. 7402 (Merrimack River-Hopkinton-Everett Res.)-41.

The proposed Beards Brook Reservoir and the Bennington Reservoir on the Contoocook River are being substituted for the previously approved Hopkinton-Everett Reservoir in accordance with the recommendations contained in the correspondence referred to in the above paragraph.

2. Investigations.-

a. Scope of Investigations and Studies.- Complete investigations and studies have been made of all factors affecting the design and construction of the reservoir. Data have been compiled, studied and analyzed on climatical, hydrological, and geological conditions, flood heights, frequencies and losses, power possibilities, economics of construction, and benefits to be derived from the construction of the project. Detailed descriptions of the investigations, analytical studies and results are contained in other sections of this report.

b. Previous Investigation of Contoocook River Basin.- A comprehensive system of flood control reservoirs and related flood control works in the Merrimack River Basin was authorized by the Flood Control Act of 28 June 1938 (Public No. 761 - 75th Congress, 3rd Session) and funds were appropriated for the work. This authorization for a comprehensive system was based on a report and recommendations made by the District Engineer which were submitted to Congress and published as House Document No. 689, 75th Congress, 3rd Session. In the preparation of this report, preliminary studies of the Beards Brook site, exclusive of the North Branch, were made and the site listed as a possible location for a reservoir, although construction was not recommended due to the estimated high cost. This report placed emphasis on the Riverhill site in the westerly part of Concord for control of the Contoocook River. Due to local opposition, the Riverhill site was abandoned and the West Peterboro, Mountain Brook and Hopkinton-Everett reservoirs were substituted.

Various reports were submitted on the three reservoirs, including a definite project report on the Hopkinton-Everett project which was approved, subject to minor modifications, by the Chief of Engineers on 12 March 1940 (File E.D. 7402 (Merrimack River, Hopkinton-Everett Reservoir)-8). During the early period of planning on this project the matter was referred to the Federal Power Commission for comment and recommendation. The Commission carried on extensive investigations of the possible over-all development of the Contoocook River Basin, for which investigations the District Engineer obtained and furnished the basic data. The report and recommendations of the Commission were presented at the time hearings were being held in response to the request by the War Department for approval by the State of New Hampshire for the acquisition of land for the Hopkinton-Everett project. The Commission's report included proposals for the construction of a series of reservoirs, including one on Beards Brook at Hillsboro Lower Village as alternates for the Hopkinton-Everett project.

The Commission's proposal resulted in a series of studies being made by the War Department and the Federal Power Commission, but since agreement as to the best means of development could not be reached, the matter was referred to the Board of Engineers for Rivers and Harbors for recommendation (letter from the Chief of Engineers, dated 6 December 1941 subject: "Reservoir Plans for the Contoocook Basin, New Hampshire", File 7402 (Merrimack River - Hopkinton-Everett Res.)-41). The recommendation of the Board of Engineers that the Beards Brook and Bennington Reservoirs be constructed in lieu of the Hopkinton-Everett Reservoir is contained in the 9th Indorsement to this letter.

c. Public Hearings.- A public hearing on the Beards Brook Reservoir was held at Hillsboro, New Hampshire, on 14 August 1945 by the Water Resources Board of the State of New Hampshire. This agency has been commissioned by state legislative action to study proposed flood control projects, hold public hearings and to make recommendations to the Governor's Council regarding such projects prior to action by the Governor's Council with regard to granting permission to the United States to acquire land and rights-of-way.

The hearing was attended largely by affected property owners and other interested local parties, approximately 75 persons attending. Questions and discussion centered largely around methods of land acquisition, flowage limits as affecting individual properties and appearance and condition of the reservoir area following flooding. The loss of taxes through Government purchase of land, highway relocations and the substitution of smaller upstream reservoirs to replace the proposed project were other topics of discussion. Considerable opposition to the construction of the reservoir was expressed at the meeting by local residents, based almost entirely on the disruption to be caused by the purchase of the land and the alleged nuisance aspect of the reservoir area after periods of heavy flooding.

3. Local Cooperation.- Local financial cooperation is not required as Section 2 of the Flood Control Act approved 28 June 1938 (Public No. 761 - 75th Congress, 3rd Session) applies to this project.

4. Definite Project Plan.-

a. General.- The proposed reservoir controls practically the entire drainage area of Beards Brook and the North Branch of the Contoocook River. The reservoir will be operated as a simple retarding basin, gate operation being used only for decreasing the time of emptying. The reservoir is contained principally within the Beards Brook basin, with a drainage area of 56 square miles, and the dam and spillway are located so that flood water from the sixty-four-square-mile drainage area of the North Branch which is spilled from the power pool of the Jackman Reservoir is diverted into the proposed reservoir. The ponded area of the reservoir at spillway crest elevation 705 is 810 acres, and the storage capacity is 35,000 acre-feet which is equivalent to 5.5 inches of run-off from the combined drainage area of 120 square miles.

b. Project Description.- The impounding structures will consist of an earth dam and a concrete ogee section spillway as shown on Plates IV-1 to IV-5 accompanying Appendix IV of this report. The dam will be of the rolled-earth embankment type, top elevation 723, with an over-all length of approximately 4,400 feet and a maximum height of 108 feet, containing approximately 1,371,000 cubic yards of earth and rock fill and will be constructed across the Beards Brook Valley. The spillway with a crest length of 450 feet at elevation 705 will be constructed on bedrock across the North Branch Valley. Normal flow in Beards Brook will pass through the outlet works which will be constructed in the approximate location of the existing Beards Brook stream bed. Normal spillage from the power pool of the Jackman Reservoir on the North Branch will pass through an ungated outlet, 4' x 6', in the spillway section. Flood flows on the North Branch in excess of the ungated outlet capacity will pass through a diversion channel into the Beards Brook Reservoir. The diversion channel excavated through a low ridge on the upstream side of the embankment, will serve also as a spillway approach channel for the spillway design flood from the reservoir. The outlet works include a reinforced concrete intake structure, 3 hydraulically-operated sluice gates, a single conduit 8' square, and a stilling basin with gravity type guide walls, all of which are founded on glacial till. A rolled-earth dike will be constructed on the west bank of the diversion channel and the north bank of the North Branch to protect the existing wood stave pipe, 7' x 6" in diameter, extending from the Jackman Reservoir to the surge tank downstream from the proposed dam. The existing highway from Hillsboro Lower Village to the town of Hillsboro, crossing the site of the proposed dam at Station 9 + 00 will be shifted and raised to elevation 723 and will cross the diversion channel on a simple steel girder bridge, supported on reinforced concrete piers. Re-routing of the highway is considered economically unfeasible. The selected spillway site on the North Branch is considered the most suitable location due to the presence of bedrock at a depth of approximately 10 feet. The major portion of the spillway approach and discharge channel is cut through rock which falls away rapidly and necessitates the use of a concrete training wall for support of the west end of the earth dam. The earth dam location takes advantage of the topography in that the high ground on either side of the Beards Brook Valley reduces the volume in the earth fill embankment to a minimum. A study of the "Record of Exploration Plans", Plates II-4 and II-5, will indicate the impervious foundation condition throughout at a depth of approximately 15 feet at the valley bottom and 5 to 8 feet on the abutments.

c. Description of Reservoir Area.- Approximately 623 acres of the proposed reservoir area are covered with a mixed growth of hard and soft woods. The more accessible areas along Beards Brook Road have been cut over in recent years and the better stands of wood growth remaining are situated along the outer edges of the proposed reservoir where the slopes make forest operations less profitable. The reservoir area, except in the valley bottom, will not be cleared of timber, and it is expected that the general appearance after construction of the dam will be about the same as at present. The greater part of the open land in the reservoir area, approximately 119 acres, is located in the north central portion, but most of the fields have been producing little more than a hay crop for a period of years.

There are eleven sets of buildings in the entire reservoir area. The most important units, consisting of village residences and subsistence homesteads are the seven properties along State Route No. 9, located within the construction area of the proposed dam, diversion channel and highway bridge approaches. The remaining four buildings within the reservoir area consist of two moderately priced summer homes, a subsistence homestead and a vacant farm.

The Public Service Company of New Hampshire has built the Jackman Hydroelectric Power Development on the North Branch of the Contoocook River. This 4000 K.W. installation includes a wood stave pipe line 7-1/2 feet in diameter, extending from the Jackman Dam 2500 feet above the Beards Brook dam to the surge tank downstream. The section of pipe line crossing the proposed dam and diversion channel, will be replaced by a steel pipe line, 778 feet long, encased in concrete, which will be placed underground as illustrated on Plate IV-8.

A water pipe line for the town of Hillsboro water supply system passes through a section of the proposed reservoir along Bible Hill Road with lowest elevation 645. Relocation of this water main is not proposed since the reservoir normally will be empty.

d. Design Floods.-

(1) Reservoir Design Flood.- The two largest floods of record on Beards Brook, occurring in March 1936 and September 1938, were utilized as reservoir design floods. The larger of these, the March 1936 flood, had an estimated peak discharge of 10,200 c.f.s. and an estimated 16-day volume of about 93,000 acre-feet, equivalent to about 14.4 inches of run-off over the watershed. The September 1938 flood had a peak discharge of 10,000 c.f.s. and an estimated 8-day volume of 48,300 acre-feet, equivalent to 7.7 inches of run-off over the watershed.

(2) Spillway Design Flood.- The storm used for the spillway design flood comprises the computed maximum possible rainfall over the 120 square-mile drainage area, or 18.4 inches of rainfall in 24 hours with 15.3 inches occurring in 6 hours. This storm was considered to be centered over the Beards Brook section of the drainage area. The computed maximum possible rainfall for this 56 square-mile section was 19.6 inches in 24 hours. The residual rainfall, distributed over the 64 square-mile North Branch drainage area, amounted to a total of 17.3 inches in 24 hours. Run-off on both the North Branch section and the Beards Brook section was computed by means of synthetic unit graphs assuming an infiltration rate of 0.05 inches per hour and a base flow of 5 c.f.s. per square mile of drainage area. The resulting spillway design flood for the Beards Brook section had a peak inflow of 57,000 c.f.s. and a volume of 18.6 inches. The corresponding figures for the North Branch showed 23,900 c.f.s. peak inflow and a volume of 16.4 inches. The combined spillway design hydrograph had a peak of 66,500 c.f.s. and a volume of 17.4 inches over the total drainage area. The maximum spillway discharge when routed through the reservoir is 62,500 c.f.s. with a maximum pool elevation of 716.0 in the North Branch section of the reservoir, and 717.2 in the Beards Brook section. It is assumed that during a flood of the proportion of the spillway design floods the Jackman dam will be over-topped and fail. Assuming this occurs, the peak spillway discharge at Beards Brook will be 79,300 c.f.s. with a resulting pool elevation in the North Branch section of the reservoir of 717.9 and 718.8 in the Beards Brook section.

e. Method of Operation.- The reservoir will be operated primarily to regulate the flood discharges of Beards Brook so as to provide maximum benefits at downstream damage centers. The basic operation will be similar to that of a simple retarding basin, gate operation being resorted to in order to decrease the time of emptying the reservoir.

f. Reservoir Capacity.- The original selected capacity of the reservoir was 38,000 acre-feet with spillway crest at elevation 703, as recommended by the Board of Engineers for Rivers and Harbors. This capacity was based on area-capacity curves computed from interpolations made from U. S. Geological Survey plans with 20-foot contour intervals. A new area-capacity curve, computed from a 5-foot contour interval survey made in 1944 by aerial survey methods indicated that the reservoir capacity was 33,000 acre-feet at the original selected spillway crest elevation 703. A detailed topographic survey of Hillsboro Lower Village disclosed that the limiting spillway crest elevation can be raised to elevation 705 without flood damage to the Village, increasing the reservoir capacity to 35,000 acre-feet. This capacity has been selected for the project plan and provides the highest ratio of benefits to costs although the estimated cost is approximately $700,000 more than the cost estimated by the Board of Engineers.

g. Alternative Layouts.- The site of the proposed dam was selected initially to take advantage of the physical and topographical features of the valley. Results of the explorations and studies made of the foundation area showed that the soil characteristics and foundation were suitable for construction of an earth embankment type dam. The alternative layouts were made at the selected site based on economic studies of the relative costs and benefits of constructing the proposed reservoir with or without increasing the spillway capacity of the existing Jackman dam, and of constructing the Beards Brook dam to control only the run-off from Beards Brook. In the event of the occurrence of the spillway design flood on the North Branch, the Jackman dam would be over-topped and would fail due to the limited capacity of that spillway. Failure of the dam would result in a maximum water surface at Beards Brook dam 1.9 feet higher than would occur if the flood were routed through the Jackman Reservoir, using an enlarged spillway capacity to eliminate the possibility of over-topping. The assumed design conditions for the alternative layouts and the results of the analyses are as follows:

(1) Alternate A.- Beards Brook dam and spillway constructed across Beards Brook and the North Branch, with spillway crest elevation 705 and top of dam elevation 723, as described in this report and assuming failure of Jackman dam. Alternate "A" was selected finally as the most economical layout.

(2) Alternate B.- Beards Brook dam located as described for Condition A. The Jackman spillway capacity enlarged to discharge the spillway design flood without over-topping the dam. This condition permits lowering the proposed dam 1.0 feet.

(3) Alternate C.- Beards Brook dam constructed to control only the Beards Brook run-off with a drainage area of 56 square miles; the spillway crest established at elevation 685 and discharging into the North Branch. Alterations to the Jackman dam have not been assumed since the Beards Brook dam would not be affected by failure of the Jackman dam under this condition.

	Alternate A Beards Brook &	Alternate B North Branch	Alternate C Beards Brook
	Without Extension of Jackman Spillway	With Extension of Jackman Spillway	Only
Net Drainage Area in Square Miles	120	120	56
Spillway Crest Elevation	705	705	685
Top of Dam Elevation	723	722	704
Reservoir Area in Acres	810	810	680
Storage Capacity in Acre-feet	35,000	35,000	20,400
Storage Capacity in Inches of Drainage Area	5.5	5.5	6.8
Estimated Cost (Including Overhead and Contingencies):			
Reservoir Costs	$ 244,000	$ 244,000	$ 97,000
Reservoir Clearing	16,000	16,000	16,000
Construction Costs	2,920,000	2,870,000	2,430,000
Jackman Dam - Extension of Spillway	-	300,000	-
Total Estimated Cost	$3,180,000	$3,430,000	$2,543,000
Cost per Acre Foot of Total Storage	$ 90.86	$ 98.00	$ 124.66
Annual Carrying Charges	142,593.	155,000.	115,000.
Annual Flood Control Benefits	114,000.	114,000.	78,000.
Ratio of Annual Benefits to Annual Carrying Charges	.80	.74	.68

5. Structures and Improvements.-

a. Embankment.- The embankment section of the dam is designed as an earth fill section consisting of a compacted impervious core backed with compacted random fill and flanked with a shell of compacted pervious fill as indicated in the sections on Plate IV-3. The upstream and downstream slopes are protected by dumped rock blankets. The earth section has a top width of 25 feet at elevation 723; an average upstream slope of 1 on 2.5; an average downstream slope of 1 on 2.25; a maximum height of 108 feet and a total length of approximately 4,400 feet. The impervious core cut-off extends into the underlying glacial till except for a short distance adjacent to the spillway where it extends to bedrock as indicated on Plate IV-1.

b. Outlet Works.- The outlet works consist of controlled and uncontrolled outlets. The discharge of Beards Brook is regulated by the controlled outlet works which include an intake structure, reinforced concrete conduit and stilling basin founded on till. The intake structure is of reinforced concrete and contains two outlets 3'-0" x 7'-0", and one outlet 4'-0" x 7'-0", each of which is equipped with a hydraulically operated sluice gate. The hydraulically operated sluice gates were selected because of the lower cost of installation and the simplicity of operation and maintenance as analyzed in Appendix IV. Access is gained to the equipment room of the gate house from the top of the dam embankment by a structural steel access bridge. The gate chamber outlets converge into an 8-foot square reinforced concrete conduit which passes through the embankment and discharges into the stilling basin. The stilling basin is composed of a reinforced concrete mat, baffles, and end sill with gravity type retaining walls, all of which are founded on a blanket of sand and gravel laid on the till foundation. The approach and discharge channels are earth cuts with riprapped side slopes and riprapped bottoms immediately upstream of the intake structure and downstream from the end sill of the stilling basin. The discharge of the North Branch is uncontrolled and passes through the ungated rectangular outlet 6'-0" x 4'-0" in the gravity spillway structure.

c. Spillway.- The spillway is located at the westerly end of the embankment and spans the North Branch Valley. The spillway is a gravity type concrete ogee section and contains the ungated outlet for passing the normal discharge from the North Branch. The crest length at elevation 705 is 450 feet. The approach channel is approximately 150 feet in length, and has a width of 450 feet at the spillway and increases in width until it joins the diversion channel. The approach channel is entirely in rock cut and a pilot channel is provided to train the normal flow of the North Branch to the ungated outlet. The floor of the approach channel slopes away from the spillway to natural ground surface in order to provide proper drainage. The discharge channel at the spillway is at elevation 695 and slopes gradually to the present ground surface. The discharge channel has a total width of 450 feet which gradually converges to a width of 340 feet in a length of approximately 400 feet. Sections of the channel not in rock cut are protected by riprap. The normal discharge of the North Branch flows in a pilot channel which empties into the natural stream bed downstream from the spillway. The spillway training wall on the north side is a gravity type concrete section, and on the south side is a natural rock cut formed by excavation of the channel.

d. Dike.- An earth fill dike, composed of compacted selected fill with a blanket of dumped rock on the upstream face, is located on the westerly side of the diversion channel and the north bank of the North Branch. This dike is provided to protect the existing wood stave pipe line which supplies water from Jackman Reservoir to an existing surge tank located at the confluence of the North Branch and Beards Brook.

e. Diversion Canal and Diversion Channel.- A diversion canal is provided upstream from the spillway to divert flood waters of the North Branch into Beards Brook Reservoir. When the flood waters reach spillway crest elevation 705, the function of the diversion canal is reversed and it guides the impounded reservoir water through the canal and diversion channel to the spillway where it is discharged into the North Branch.

The diversion canal, having a trapezoidal cross-section with a bottom width of 90 feet, is approximately 1,100 feet long with the invert varying from elevation 689 near the spillway to elevation 686 at the control weir. (See Plate IV-2.) The side slopes and bottom of the canal have hand-placed riprap. A control weir, with crest elevation 690 and length of 110 feet is provided at the end of the canal to control the velocity of flow in the canal and eliminate scouring and erosion. The discharge over the control weir will be dissipated in a dumped rock paved area before flowing overland to the reservoir pool.

The diversion channel, with a minimum bottom width of 300 feet, is superimposed on the diversion canal. It is large enough to pass the spillway design flood with a velocity of approximately 8 feet per second.

A new steel girder highway bridge, constructed on concrete piers, spans the diversion channel.

f. Operator's Quarters.- It is proposed to construct a wood frame house with finished interior to serve as an office and as living quarters for the operator of the dam. This house is to be located in the area between the highway and the dike westerly of the diversion canal.

6. Foundations.-

a. Results of explorations and studies indicate that a satisfactory foundation for the embankment and appurtenant structures exists at the selected site. Previous to glaciation, the drainage channel of the Beards Brook area was a bedrock valley consisting of granite porphyry with many schist inclusions. During the period of glaciation, deposits of compact sandy and silty till covered the bedrock and existing gravel and sand. Within the original valley there now exists two streams: Beards Brook, located almost directly above the original valley bottom, and the North Branch of the Contoocook River, located high on the west wall. Bedrock is exposed only at the location proposed for the spillway of the dam where the North Branch has eroded the overburden.

b. The existing soils consist of gravelly sands and glacial till. The sands occur principally in a shallow surface layer resulting from weathering of the till or from post-glacial deposition. A local deposit of sand, probably of pre-glacial origin, was encountered between the till and bedrock in the bottom of the bedrock valley. The principal soil body is compact sandy and silty till of low permeability and high shearing strength.

c. A summary of the foundation investigations and analyses is presented in Appendix II of this report.

7. Hydroelectric Power.- The Board of Engineers for Rivers and Harbors, in 9th Indorsement, dated 8 November 1943, subject: "Reservoir Plans for the Contoocook Basin, New Hampshire", recommended consideration of Beards Brook site in the interests of flood control and for storage for stream regulation at some future time. Results of studies conducted to this end are presented in Appendix V and indicate that such multiple-purpose development of the site is not economically justified.

8. Relocations.- The construction of the reservoir will necessitate raising short lengths of two roads, the improvement of an existing cemetery access road, the construction of two highway bridges, and the raising or relocation of a small number of utilities.

State Highway No. 9, a first class bituminous-surfaced highway, which connects the towns of Hillsboro and Hillsboro Lower Village, passes through the site of the earth embankment and diversion canal. It is proposed to span the diversion canal with a new steel girder highway bridge and raise the approaches of the existing road.

That portion of the Antrim-Hillsboro Lower Village gravel-surfaced road where it crosses the North Branch is subject to inundation and the existing stone double-arch bridge is inadequate in area for passing flood stage flows. A new highway bridge, designed to pass a flood of the 1936 magnitude, is proposed to replace the existing bridge, with the deck and adjacent approaches raised above spillway crest elevation, as indicated on Plate V-1.

Access to the Bible Hill Cemetery, which is located upstream from the east abutment of the dam, is gained from either the Beards Brook Road or Bible Hill Road at the present time. Access passage to the cemetery via Beards Brook Road will be blocked by the dam. Therefore, it is proposed to widen and surface the existing access roadway from Bible Hill Road to provide access to the cemetery at all times.

Raising State Highway No. 9 and the Antrim-Hillsboro Lower Village Road also requires the raising of the power and telephone service lines adjacent to the roads.

A wood stave pipe line which supplies water from the Jackman Reservoir to the surge tank located at the confluence of the North Branch and Beards Brook passes through the site of the proposed diversion canal and earth embankment. It is proposed to lower this pipe line below the diversion canal from the proposed dike at the bank of the diversion channel to a point approximately 200 feet downstream from the toe of the embankment and replace the existing wood pipe with a steel-lined concrete conduit, as indicated on Plates IV-2 and IV-8.

The proposed methods for accomplishing the relocations, including the new construction involved, and the views of the owners, are described in Appendix VI.

9. Availability of Construction Materials.-

a. All suitable materials removed from required structure excavations at the site are scheduled for use in the embankment. It is proposed to use silty glacial till from a borrow area on the east wall of the valley for the impervious core material. Gravelly sand for pervious fill is available from a selection of borrow sources within 1 to 2 miles upstream from the dam site. Rock for slope protection will be available from bedrock excavation at the spillway site and from cobbles and boulders removed from the required earth excavations.

b. Summaries of the materials available from excavation with indicated disposition, and materials required for construction with their proposed source, are presented in the tabulations given in paragraph c of Appendix II.

10. Construction Time Required and Schedule of Operations.-

a. Required Construction Time.- The construction of the dam and spillway is scheduled to be accomplished by contract during the summer months over a three-year period, due to the severe winter weather in this area.

b. Schedule of Operations.- The schedule of construction operations, based upon completion of the dam and appurtenant works in 3 years as illustrated on Plate IV-11, is as follows:

First Season.- Clear and strip the dam site, spillway and diversion channel area; construct the 7'-6" diameter pipe line and adjacent cofferdam; excavate diversion canal and portion of diversion channel; construct portion of spillway to elevation 695; construct entire retaining wall and filled area north of spillway discharge channel; construct conduit stilling basin and walls, approach channel walls; construct intake structure to elevation 650; excavate portions of approach and discharge channel; construct highway bridge and approaches for highway Route No. 9; construct embankment to elevation 705 from Sta. 8 + 30 to Sta. 34 + 00.

Second Season.- Divert Beards Brook through the conduit; construct cofferdams across Beards Brook; complete the spillway, spillway approach and discharge channels, diversion channel and diversion canal; complete the intake structure and dam embankment; construct dike along diversion channel.

Third Season.- Construct access bridge to gate house; final grading and placing of topsoil; construct operator's quarters; raise Antrim Road; construct new bridge and relocate other facilities in the reservoir area.

c. Funds Required by Fiscal Years.- The funds required during each of the three year construction periods, for the accomplishment of the project, including land acquisition and relocations, construction operations, and engineering, contingencies and overhead, are estimated to be as follows:

First Year	$ 1,300,000.
Second Year	1,600,000.
Third Year	280,000.
	$ 3,180,000.

The estimated funds requirements are based upon the execution of all construction by contract work, except that the work scheduled to be accomplished by the owners will be done on a reimburseable basis as outlined in Appendix VI.

d. Preparation of Plans and Specifications.- Prior to preparation of final plans and specifications, further explorations of foundation and borrow materials are required. The estimated cost of the additional explorations is approximately $ 8,000. It is estimated that a period of six months will be required to prepare the contract plans and specifications, at an estimated cost of $40,000.

e. Employment Analysis.- Based on the average factors contained in the Bureau of Labor Statistics estimates as transmitted by letter dated 17 January 1945 from the Chief of Engineers to the Division Engineer, New England Division, subject: "On-site and off-site Labor Estimates for Inclusion in Definite Project Reports", it is estimated that the project reported herein will provide the following number of man hours of labor:

On-site Labor	
Skilled	280,000 man hours
Unskilled	1,100,000 " "
Other	120,000 " "
Total	1,500,000 " "
Off-site Labor	1,600,000 " "
Total Labor	3,100,000 " "

11. Clearance with Other Agencies.- In accordance with Circular Letter No. 2652, dated 1 January 1944, subject: "Quadripartite Agreement" and Circular Letter No. 2306, dated 1 August 1944, subject: "Fish and Wild Life Service Cooperations", information and pertinent data on the construction of the proposed Beards Brook Reservoir have been imparted to the Soil Conservation Service of the Department of Agriculture; the Federal Power Commission; and the Fish and Wild Life Service of the Department of the Interior. The Directors of the several state agencies responsible for the promotion of public recreational facilities in the State of New Hampshire have been consulted; namely, the State Planning and Development Commission and the Forestry and Recreation Department. Regarding their possible interest in the development of recreational facilities in the Beards Brook Reservoir, the directors of both agencies exhibited a definite interest in the development of a recreational lake in accordance with description and plans shown on Appendix VII. Contacts have also been made with the Fish and Wild Life Service of the Department of the Interior and the Fish and Game Department of the State of New Hampshire regarding the interest of these agencies in the reservoir. Plans and other information have been furnished at their request, and it is understood that a complete investigation and study is being made of the project by these services. In accordance with the provisions of the Flood Control Act approved 22 December 1944 (Public Law 534-78th Congress - 2nd Session) and as this project in its entirety lies east of the ninety seventh meridian, the Department of the Interior has not been consulted with

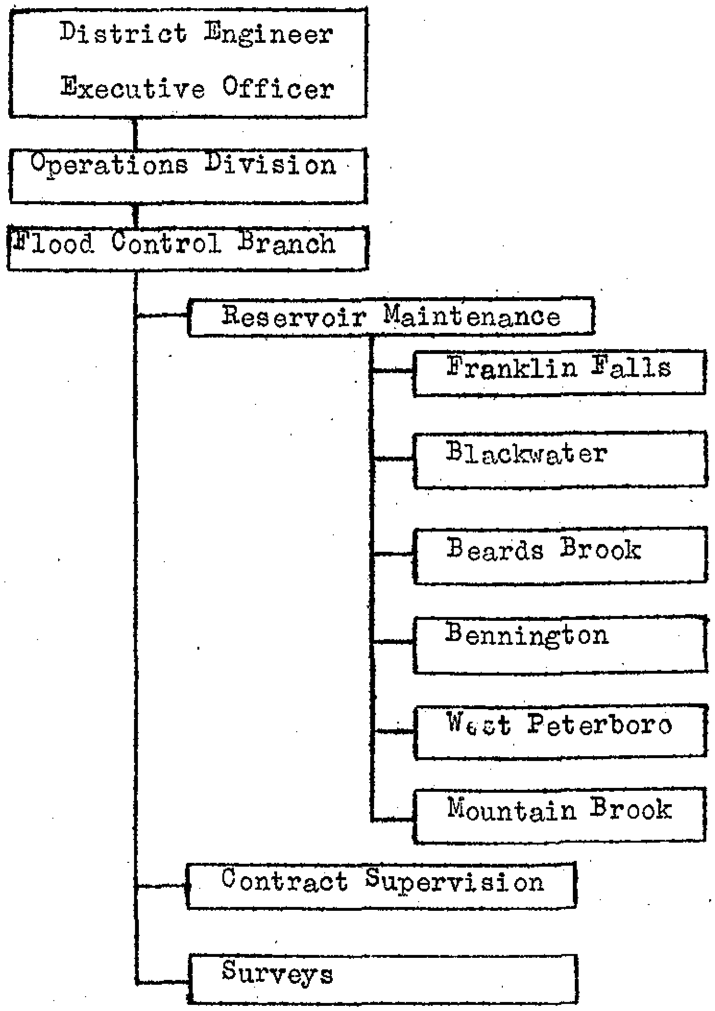
reference to matters of land reclamation.

The Soils Conservation Service, Department of Agriculture, has been informed of the proposed flood control program being planned by the Boston District Office for co-ordination of planning for soil conservation in the development of the projects. The Department of Agriculture acknowledged receipt of the data furnished and forwarded its "Report - Soil Erosion Conditions in New Hampshire", for review, and wishes to be informed of any hearings which may be held. Further action by the Department of Agriculture has not been indicated to date.

12. Operation and Maintenance.- The operation and maintenance of the dam will be a Federal function. The operational duties will consist of operation of the gates during and after flood periods. Maintenance will consist of upkeep of equipment in the operating house, maintenance of the operating house and dam structure, including slopes, drains, and access road. Removal of debris at outlets and policing of the reservoir area will be a part of the maintenance duties of the operation and maintenance personnel.

The operation and maintenance functions at the Beards Brook dam will be performed under the general aupervision of the Superintendent assigned to the Bennington dam located approximately 10 miles south of the Beards Brook dam. The permanent organization at the dam will consist of one classified dam tender, CPC-6, $2,166, who will be provided with quarters at the site and one unclassified laborer.

The operation and maintenance force will be under the supervision of the Flood Control Branch of the Operations Division of the Boston District, U. S. Engineer Office. The organizational structure of the operational and maintenance force is shown on the following chart.



The estimated annual cost of maintenance and operation is as follows:

Item of Work	Estimated Cost
Services of dam tender and laborer	$ 4,200.00
Power, lubrication and lubricants	150.00
Heating operating structure	250.00
Clearing and removal of debris from Reservoir area	2,500.00
Landscaping, reseeding and mowing	1,000.00
Maintenance of access roads, surface drains and slopes	1,000.00
Repairs and painting	700.00
Miscellaneous	500.00
District Office overhead	700.00
Total	$11,000.00

13. Malaria Control.- Action has been initiated to determine the need and requirements for malaria control in the reservoir area in accordance with the instructions contained in Circular Letter No. 3606, dated 9 March 1945, subject: "Malaria Control at River and Harbor and Flood Control Reservoirs." The subject has been discussed with representatives of the U. S. Public Health Service, and plans and pertinent data relative to the reservoir have been forwarded to that office. The area has been inspected and studied by a representative and an etymologist of the U.S. Public Health Service in company with a representative of the Boston District Office. It is the informal opinion of the U. S. Public Health Service that malaria control in the reservoir area is a problem of only minor consideration. A formal report on the requirements for malaria control resulting from the study made has not been received to date. When the recommendations of the U. S. Public Health Service are received, the information will be included in this report as a supplementary appendix.

14. Cost Estimate.-

a. Total Costs.- The estimated total costs, including engineering, contingencies and overhead for the proposed reservoir are based upon the estimated quantities and unit prices itemized in the following table. The estimated costs of lands and damages are based upon an appraisal made in September 1945 by the Boston District Real Estate Suboffice, in conjunction with representatives of this office and are an equitable evaluation of the items involved.

DETAILED ESTIMATE OF COST

I. RESERVOIR COSTS	Total Cost
Land and Improvements for Reservoir) Work and Borrow areas incl. riparian rights)..	$120,000.
Highway, Telephone and Power Line relocations	73,000.
Sub-total - Reservoir Costs....................	$193,000.
Contingencies (15%)........................	28,950.
	$221,950.
Government Expenses (10%) ±)................	22,050.
TOTAL RESERVOIR COSTS	$244,000.

II. CONSTRUCTION COSTS

	Quantity	Unit	Unit Price	Total Cost
Removal of existing structures	–	L.S.	–	$ 2,000.
Stream Diversion and Pumping	–	L.S.	–	20,000.
Clearing and grubbing	60	Acres	$300.00	18,000.
Stripping – earth	137,000	C.Y.	.50	68,500.
Stripping – rock boulders	8,000	C.Y.	3.00	24,000.
Excavation – earth	243,000	C.Y.	.50	121,500.
Excavation – rock boulders	17,000	C.Y.	3.00	51,000.
Excavation – rock ledge	75,000	C.Y.	2.50	187,500.
Borrow, Pervious	640,000	C.Y.	.50	320,000.
Borrow, Impervious	450,000	C.Y.	.55	247,500.
Random fill from borrow area	120,000	C.Y.	.55	66,000.
Rolled fill-Pervious	460,000	C.Y.	.12	55,200.
Rolled fill-Impervious	290,000	C.Y.	.15	43,500.
Rolled Fill -Random	280,000	C.Y.	.12	33,600.
Rolled Fill – Semi-compacted	40,000	C.Y.	.10	4,000.
Structure backfill	10,000	C.Y.	.60	6,000.
Selected Gravel	115,000	C.Y.	1.50	172,500.
Screened Gravel	2,000	C.Y.	2.00	4,000.
Dumped Riprap (Transportation and placing only)	185,000	C.Y.	.60	111,000.
Hand placed Riprap (Transportation and placing only)	14,000	C.Y.	3.00	42,000.
Derrick Stone	1,000	C.Y.	5.00	5,000.
Topsoil and organic subsoil	10,000	C.Y.	2.50	25,000.
Concrete Gate Structure and Conduit	3,000	C.Y.	20.00	60,000.
Concrete Intake Walls and Stilling Basin	4,000	C.Y.	15.00	60,000.
Concrete Spillway and Retaining Wall	7,800	C.Y.	15.00	117,000.
Concrete Diversion Canal Spillway and Walls	1,800	C.Y.	16.00	28,800.
Concrete around 7'-6" steel pipe	2,600	C.Y.	15.00	39,000.
Concrete for Bridge Piers	2,600	C.Y.	18.00	46,800.

II. CONSTRUCTION COSTS (Continued)

	Quantity	Unit	Unit Price	Total Cost
Reinforcing Steel	1,500,000	Lbs.	.06	$ 90,000.
Steel lining for 7'-6" Diam. Pipe 1/4" thick-778' long	200,000	lbs.	.14	28,000.
Gates, hoists and mechanical equipment		L.S.		38,500.
Gate House Superstructure		L.S.		10,000.
Access Bridge to Gate House		L.S.		9,000.
Highway Bridge across Diversion Channel		L.S.		100,000.
Drainage		L.S.		6,000.
Bituminous surface treatment	11,000	S.Y.	.35	3,850.
Bituminous concrete pavement 1-1/2"	500	S.Y.	.80	400.
Bituminous concrete pavement 3"	5,600	S.Y.	1.50	8,400.
Operator's Quarters		L.S.		8,000.
Miscellaneous Items		L.S.		54,450.
Sub-total - Construction Costs				$2,336,000.
Engineering, Inspection, Overhead and Contingencies (25%)				584,000.
TOTAL CONSTRUCTION COSTS				$2,920,000.

III. CLEARING COSTS

Reservoir Clearing	12,500.
Government Expenses (25%)	3,500.
TOTAL CLEARING COSTS	$16,000.

IV. SUMMARY

Reservoir Costs	244,000.
Construction Costs	$2,920,000.
Clearing Costs	16,000.
TOTAL ESTIMATED COST	$3,180,000.

V. UNIT STORAGE COST:

$$\frac{3,180,000}{35,000} = \$90.86 \text{ per acre foot}$$

b. Carrying Charges.- The total annual carrying charges for the reservoir, based upon interest on the investment, on amortization of structures and equipment, and on operation and maintenance, is $142,593, as summarized in the following table.

ANNUAL COSTS AND CARRYING CHARGES

FEDERAL INVESTMENT

1. Total First Cost:

a. Structures and Improvements with 50 year life ...	$ 3,055,000
b. Equipment with 25 year life...	125,000
c. Total...	$ 3,180,000

2. Interest During Construction:
(3% for one-half construction period)

a. On structures and Improvements with 50 year life	137,475
b. On equipment with 25 year life...	5,625

3. Total Investment:

a. Structures and Improvements with 50 year life...	$3,192,475
b. Equipment with 25 year life...	130,625
c. Total Federal Investment...	$3,323,100

ANNUAL FEDERAL CARRYING CHARGES

1.	Interest on Investment @ 3%...	$ 99,693
2.	Amortization:	
	a. Structures and Improvements with 50 year life (0.887%)...	28,317
	b. Equipment with 25 year life (2.743%)...	3,583
3.	Operation and Maintenance ...	11,000
4.	Total Annual Federal Carrying Charge...	$142,593

Construction Period - 3 years

15. Economic Study.- The proposed reservoir will increase the total benefits accruing to the comprehensive flood control system for the Merrimack River. The ratio of Annual Benefits to Annual Carrying Charges for the system as a whole is 1.31 as shown in the accompanying table, "Summary of Benefits and Costs".

The annual benefits attributable to Beards Brook Reservoir have been evaluated by computing its proportion of the discharge reduction of the March 1936 flood at the downstream reaches which would be afforded by the comprehensive reservoir system. The ratio of the proportional Beards Brook reduction to the summation of all volume reductions for reservoirs in the comprehensive system was used to determine the amount of the total annual benefits attributable to Beards Brook Reservoir for each damage center. The proportional annual benefits computed by the above method are $114,000 or a ratio of annual benefits to annual carrying charges of .80.

The summation of benefits and costs for all projects included in the present comprehensive plan for flood control of the Merrimack River is presented in the following table:

SUMMARY OF BENEFITS AND COSTS

1. Construction Costs:

a. Beards Brook Reservoir	$ 3,180,000
b. Other Flood Control Reservoirs (x)	15,117,000
c. Local Protection Projects (xx)	1,360,250
Total Construction Costs	$19,657,250

2. Annual Carrying Charges:

a. Beards Brook Reservoir	142,593
b. Other Flood Control Reservoirs (x)	698,536
c. Local Protection Projects (xx)	66,600
Total Annual Carrying Charges	$ 907,729

3. Total Annual Benefits:
Based on comprehensive flood control program including reservoirs and local protection..... $ 1,183,000

4. Ratio of Benefits to Carrying Charges:
Ratio of total Annual Benefits to Annual Carrying Charges.......... 1.31

(x) Includes completed reservoirs at Franklin Falls and Blackwater and proposed reservoirs at West Peterboro, Mountain Brook and Bennington.

	Cost	Annual Charges
Franklin Falls	$ 7,883,000–Completed	$ 372,000
Blackwater	1,280,000–Completed	65,000
West Peterboro	1,394,000–Estimated	60,518
Mountain Brook	560,000–Estimated	24,000
Bennington	4,000,000–Estimated	177,018
	$15,117,000	$698,536

(xx) Includes completed local protection project at Lowell and proposed projects at North Andover and Lawrence, Mass., and Nashua, New Hampshire.

	Cost	Annual Charges
Lowell, Mass.	$ 490,600–Completed	$ 28,300
North Andover, Mass.	323,400–Estimated	13,880
Lawrence, Mass.	329,250–Estimated	14,180
Nashua, New Hampshire	217,000–Estimated	10,240
	$ 1,360,250	$ 66,600

16. A Discussion of Factors Related to the Consideration of the Reservoir.-

a. Selection of Reservoir.- This reservoir, together with the proposed Bennington Reservoir, constitutes a substitute for the previously proposed larger Hopkinton-Everett Reservoir for control of floods on the Contoocook River. This substitution was recommended by the Board of Engineers for Rivers and Harbors upon prior recommendation of a special board of consulting engineers appointed to investigate the relative merits of the Department's plan and the plan proposed by the Federal Power Commission for control of floods arising on the Contoocook River.

b. Cost of Reservoir.- A number of circumstances have arisen to increase the cost of the Beards Brook Reservoir from a figure of $2,610,000 as reported by this office to the board of consulting engineers, to the present estimated cost of $3,180,000. Chief among these factors are the following:

(1) It has been necessary to raise the spillway and dam 2 feet to obtain a satisfactory amount of storage, (35,000 acre feet), detailed surveys indicating less storage in the area than originally determined from U.S.G.S. maps.

(2) More severe hydrological requirements consisting of higher rainfall values for the spillway design flood furnished subsequently by the Chief of Engineers and the necessity of assuming the failure of the Jackman Dam, have resulted in increasing the height of the dam an additional foot and lengthening the spillway from 340 feet to 450 feet.

(3) A sharp rise has occurred in construction costs from those prevailing prior to the war.

c. Economics of Reservoir.- The resulting present economic ratio of this reservoir is 0.8; this ratio is obtained by using annual charges based on the current estimated cost, and annual benefits based on damage figures of the 1936 and 1938 floods without adjustment to reflect the rise in prices during the war period. It is believed probable that these higher labor and price levels will prevail for an extended period, which, if true, would constitute justification for increasing the flood benefits and at the same time, the economic ratio of the Beards Brook Reservoir. The overall economic ratio of the comprehensive plan for flood control in the Merrimack Basin is 1.31.

d. <u>Approval to Acquire Land</u>.- Definite action by the State of New Hampshire has not been taken relative to request by the Department for approval to acquire land for the Beards Brook and Bennington Reservoirs. Hearings in the localities of the reservoir sites were held by the Water Resources Board of the State in the month of August. Opposition by local residents was expressed at these hearings and letters of a similar nature were submitted to the Resources Board subsequent to the hearings. At a conference of the New England Governors held in Hartford, Connecticut, on 23 October 1945, the necessity for further reservoir control on the Connecticut and Merrimack Rivers was stressed by the Governors of the States bordering the downstream reaches of these rivers, and it was agreed by the several Governors that an Interstate Flood Control Committee would be formed to work on such problems as compensation by interstate action to affected localities for loss of taxes resulting from land takings in reservoir areas. It is expected that the action taken at this conference will result in a further study by the State of New Hampshire of the request for approval to acquire land.

e. <u>Other Reservoir Studies</u>.- This office is undertaking at this time a thorough review of possible reservoir sites, particularly on tributary rivers of the Merrimack outside of the Contoocook Basin. This study will be incorporated in the pending report on Power Navigation and Flood Control in the Merrimack Basin which will be forwarded on 1 May 1946. This office will confer at an early date with the Water Resources Board of the State of New Hampshire to obtain their suggestions on flood control sites acceptable to the State. This investigation may serve to present one or more suitable and economic reservoir sites which can be proposed to supplement the lack of sufficient control resulting from the substitution of the Bennington and Beards Brook Reservoirs for the larger Hopkinton-Everett Reservoir. It may possibly disclose an acceptable reservoir located on some river other than the Contoocook which would afford protection equivalent to the Beards Brook Reservoir and at the same time would be subject to construction at a lower cost.

17. <u>Recommendation</u>.- In view of the factors discussed in the preceding paragraph, it is recommended that authorization for the construction of the Beards Brook Reservoir for the control of floods arising on Beards Brook and the North Branch be deferred, definite favorable action on this project to be contingent upon the following:

a. That the State of New Hampshire approves the Department's request for permission to acquire the necessary land for this reservoir, and

b. That further studies now being initiated do not disclose other potential reservoirs equally suitable to the Beards Brook Reservoir which could be constructed at a lower cost.

C. T. HUNT
Colonel, Corps of Engineers
District Engineer

SECTION D

APPENDICES

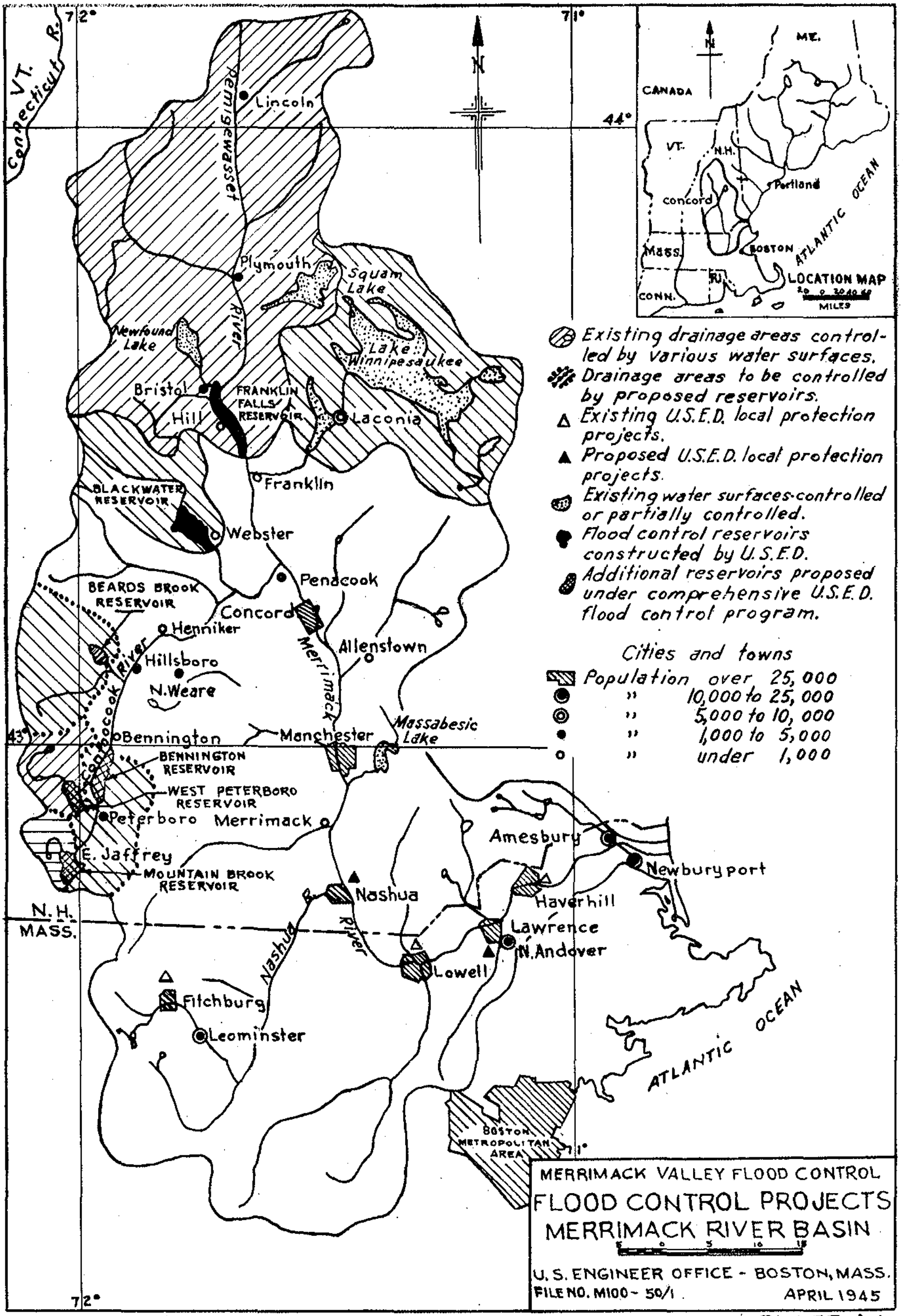
GENERAL PLANS OF PROJECT

To accompany definite project report
dated November 1945

DEFINITE PROJECT REPORT

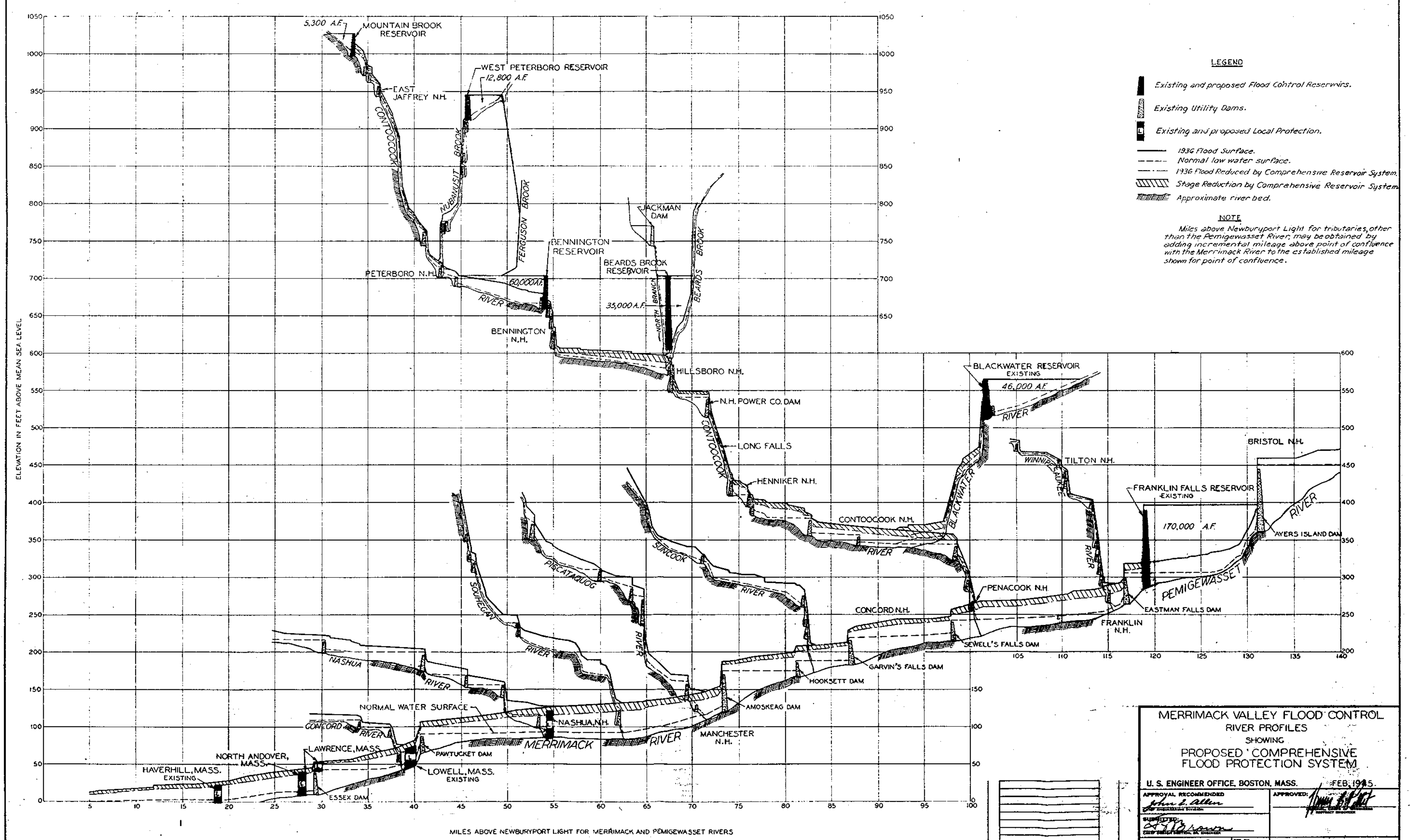
BEARDS BROOK RESERVOIR

GENERAL PLATES



MERRIMACK VALLEY FLOOD CONTROL
FLOOD CONTROL PROJECTS
MERRIMACK RIVER BASIN
U.S. ENGINEER OFFICE - BOSTON, MASS.
FILE NO. M100 - 50/1
APRIL 1945
LOCATION MAP
CANADA
ME.
VT.
N.H.
Portland
Concord
Mass.
BOSTON
CONN.
R.I.
ATLANTIC OCEAN
MILES
Existing drainage areas controlled by various water surfaces.
Drainage areas to be controlled by proposed reservoirs.
Existing U.S.E.D. local protection projects.
Proposed U.S.E.D. local protection projects.
Existing water surfaces-controlled or partially controlled.
Flood control reservoirs constructed by U.S.E.D.
Additional reservoirs proposed under comprehensive U.S.E.D. flood control program.
Cities and towns
Population over 25,000
" 10,000 to 25,000
" 5,000 to 10,000
" 1,000 to 5,000
" under 1,000
VT.
Connecticut R.
72°
71°
44°
43°
N
Lincoln
Pemigewasset
River
Plymouth
Squam Lake
Newfound Lake
Lake Winnipesaukee
Bristol
Hill
FRANKLIN FALLS RESERVOIR
Laconia
Franklin
BLACKWATER RESERVOIR
Webster
Penacook
BEARDS BROOK RESERVOIR
Concord
Henniker
Hillsboro
N. Weare
Allenstown
Merrimack
Contoocook River
Bennington
Manchester
Massabesic Lake
BENNINGTON RESERVOIR
WEST PETERBORO RESERVOIR
Peterboro
Merrimack
E. Jaffrey
MOUNTAIN BROOK RESERVOIR
Amesbury
Newburyport
Nashua
Haverhill
N.H.
MASS.
Lawrence
N. Andover
Nashua
River
Lowell
Fitchburg
Leominster
ATLANTIC OCEAN
BOSTON METROPOLITAN AREA

PLATE A-1



MERRIMACK VALLEY FLOOD CONTROL
RIVER PROFILES
SHOWING
PROPOSED COMPREHENSIVE FLOOD PROTECTION SYSTEM

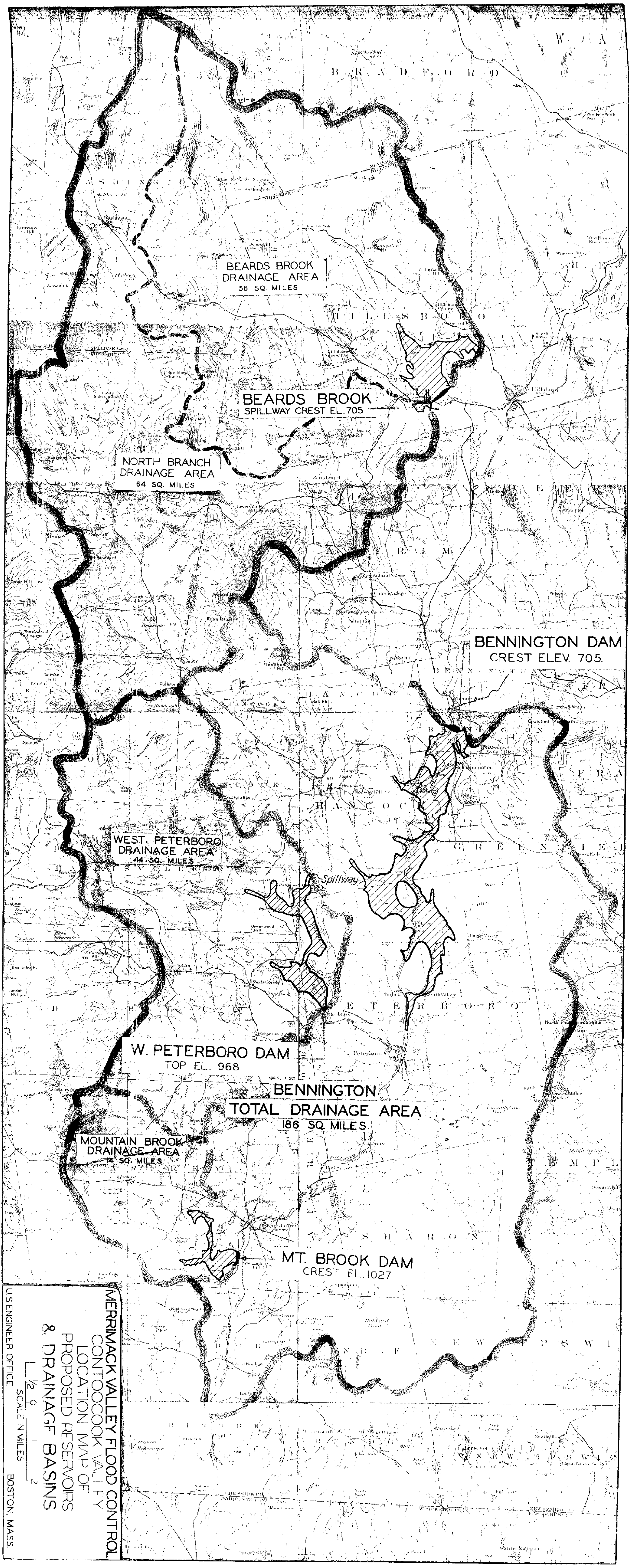
U. S. ENGINEER OFFICE, BOSTON, MASS. FEB. 1945.

APPROVAL RECOMMENDED: John L. Allen	APPROVED:
SUBMITTED: A. F. Brown	
Elliot F. Childs	FILE NO. M100-50/2

NO.	CHARACTER	DATE	BY

REVISIONS

PLATE A-2

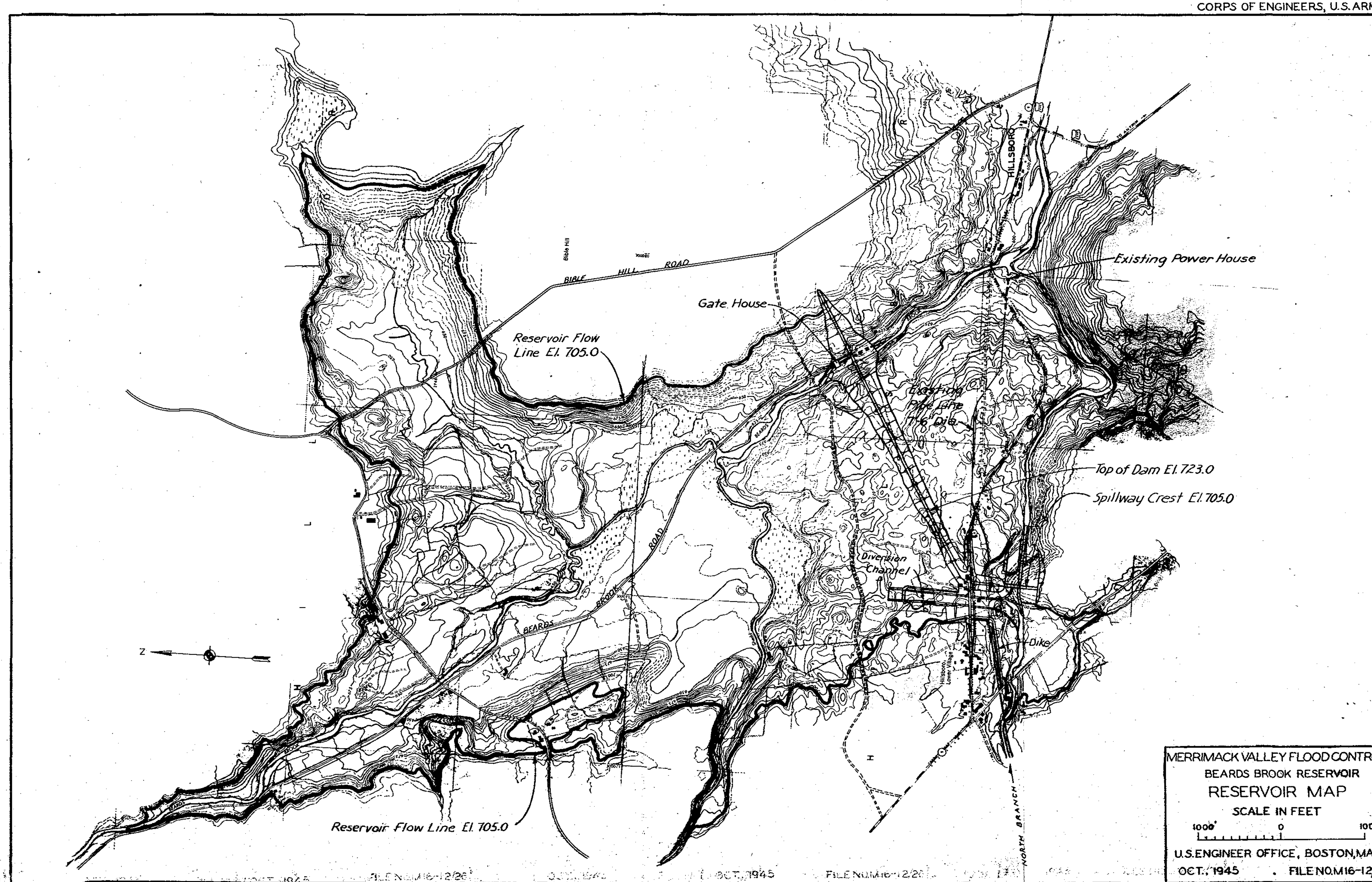


MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK VALLEY
LOCATION MAP OF
PROPOSED RESERVOIRS
& DRAINAGF BASINS

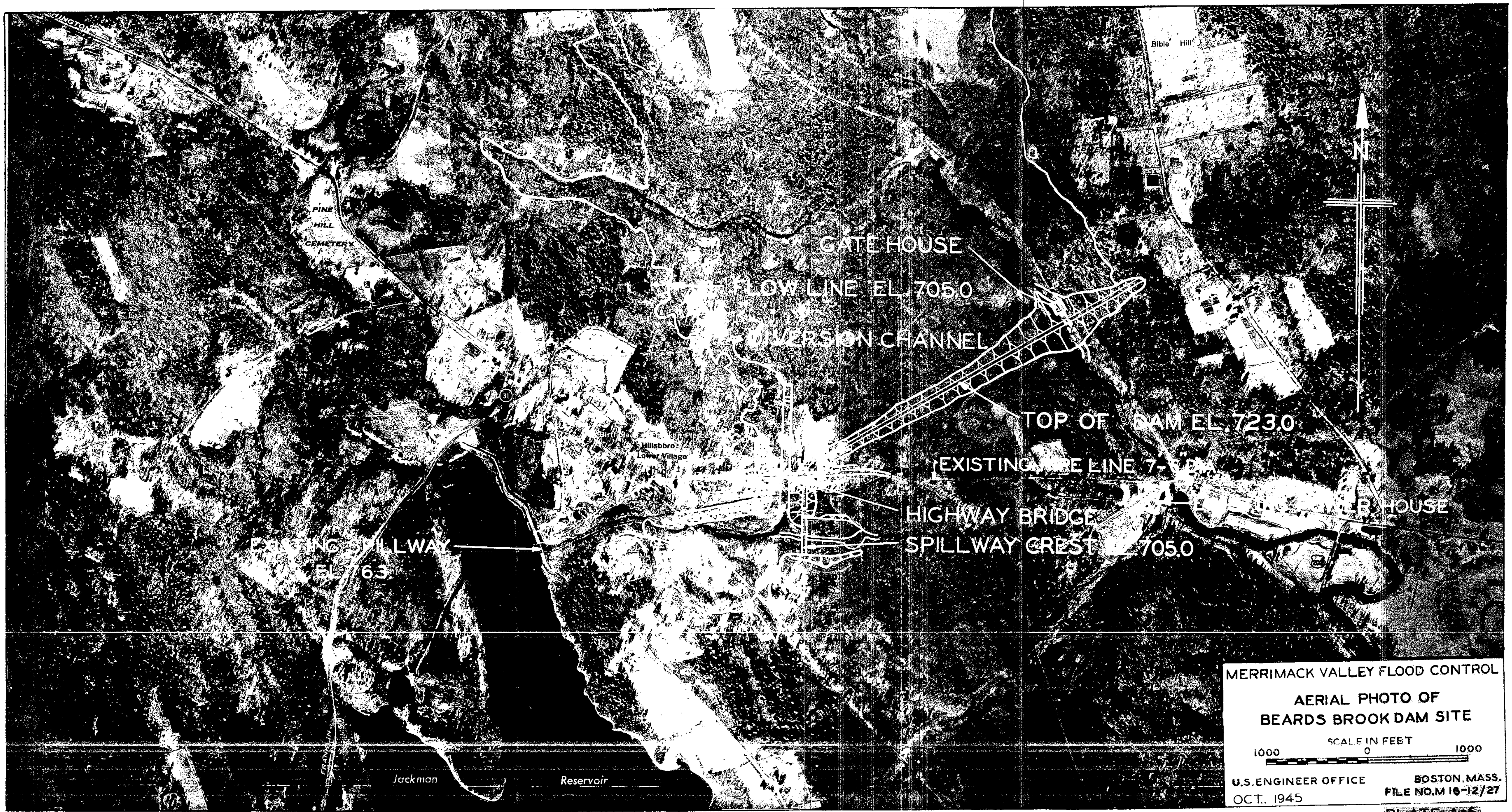
SCALE IN MILES

U.S. ENGINEER OFFICE BOSTON, MASS.

FILE NO. M 150-50/3 PLATE A-3



MERRIMACK VALLEY FLOOD CONTRO
BEARDS BROOK RESERVOIR
RESERVOIR MAP
SCALE IN FEET
1000 0 100
U.S. ENGINEER OFFICE, BOSTON, MA
OCT., 1945 . FILE NO. M16-12/



MERRIMACK VALLEY FLOOD CONTROL

AERIAL PHOTO OF
BEARDS BROOK DAM SITE

SCALE IN FEET
1000 0 1000

U.S. ENGINEER OFFICE BOSTON, MASS.
OCT. 1945 FILE NO. M 16-12/27

PLATE A-5

APPENDIX I

HYDROLOGY

To accompany definite project report
dated November 1945

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX I - HYDROLOGY

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DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX I

HYDROLOGY

a. Introduction.- The hydrological investigations, data and results of studies reported in the appendix are the basis for the hydraulic design of the Beards Brook Reservoir. The analyses include the determination of the normal discharge, the spillway capacity and crest elevation, storage requirements, the reservoir design flood, the plan of operation, the required height of cofferdam during construction and the flood stage frequencies in the reservoir.

The investigations to determine the spillway capacity and surcharge storage requirements were made in accordance with the procedure outlined in Engineer Bulletin R&H No. 9, 1938. The method involves an estimate of the maximum possible flood on the basis of available information on rainfall intensities, infiltration losses, and run-off characteristics of the drainage basin. The run-off studies consist largely of extrapolations from short records, guided by rational analyses, and the derivation of unit-hydrographs by the method developed by Franklin F. Snyder and published in the Transactions of the American Geophysical Union, Part I, 1938, pp 447-454. The computed spillway flood hydrographs were derived from the unit-hydrographs and the maximum possible precipitation values and are used for determining the spillway requirements. The two largest floods of record on the North Branch and Beards Brook were utilized as reservoir design floods.

Results of the investigations indicate that the spillway with crest elevation 705 should have a capacity of 79,300 c.f.s.; that the top elevation of the dam should be 723 M.S.L. to provide for the spillway design flood plus the discharge from the assumed failure of an upstream dam, and freeboard requirements; and that the storage capacity of 35,000 acre-feet afforded by the proposed dam and spillway, will provide effective control of all floods up to and equal in magnitude to the greatest flood of record.

b. History of Floods.- Records of great floods on Beards Brook are incomplete. Records are available for the North Branch of the Contoocook near Antrim (Drainage area = 54.8 square miles) from August 1924 to the present time. During this period of record two notable floods occurred, namely the March 1936 flood and the September 1938 hurricane and flood. Hydrographs of both of these floods have been reconstructed from available data and are shown on Plates I-24 and I-25. The observed hydrograph for the March 1936 flood at the U.S.G.S. gage on the North Branch of the Contoocook River near Antrim, New Hampshire had a peak discharge of 4680 c.f.s. and the September 1938 hydrograph had a peak discharge of 3850 c.f.s. These values were increased in proportion to drainage area to obtain the flow at the dam site. The concurrent hydrographs for the Beards Brook section were constructed from run-off and volume studies and peak discharge records for adjacent stations which resulted in an estimated peak discharge of 6300 c.f.s. for the 1936 flood, and 6000 c.f.s for the 1938 flood. When these component hydrographs were added together, the total hydrograph at the dam site for the March 1936 flood had a peak discharge of 10,200 c.f.s. and the September 1938 flood had a peak discharge of 10,000 c.f.s.

c. Watershed.- (1) Description of Basin.- The Beards Brook Dam site is located about 1-1/2 miles upstream from the confluence of the North Branch and the Contoocook River. The proposed reservoir has a tributary drainage area of 120 square miles divided into two distinct sections, namely the North Branch of the Contoocook and the Beards Brook Branch (See Plate A-3). The North Branch section consists of a long, narrow, crescent-shaped drainage area of 64 square miles, controlled by Highland Lake and Jackman Reservoir which serve as storage basins for the Jackman Hydro-Electric Development of the New Hampshire Public Service Company. The land is hilly and covered with dense secondary growth. Beards Brook section is a fan shaped area of 56 square miles, and is drained by Beards, Shedd and Woodward Brooks. The land is hilly and covered with secondary growth. Elevations in the basin range from 620 to 2480 M.S.L.

(2) Natural Storage.- There is considerable natural storage on the North Branch section of the drainage basin. This storage is distributed among Half Moon Pond, Philbrook Pond, Highland Lake, Island Pond, Robb Reservoir, and Jackman Reservoir. It is to be noted on Plate A-3 that a natural storage valley extends nearly 10 miles from the northern end of Highland Lake southerly to Robb Reservoir and Rye Pond with a difference in elevation of only 25 feet. There is not sufficient survey data available to evaluate with any accuracy the total volume of this storage, but it is large for the size of the drainage area. The only appreciable storage on the Beards Brook section is in Loon Pond, Island Pond and Contention Pond, and it is relatively small in comparison with the size of the drainage area.

(3) Stream Characteristics.- The North Branch section of the drainage area produces a slow irregular run-off, because of the large amount of natural and regulated storage. The regulated draw-down affects the shape and peak timing of the hydrograph at the

proposed dam site. The long crescent shaped drainage area tends to produce a slow sluggish hydrograph and this effect is attenuated by storage regulation for the Jackman Hydro-Electric Development downstream. The Beards Brook portion of the drainage area is drained by a three-branch stream the upper reaches of which have steep slopes characteristic of mountain streams. The little natural storage available on this stream is located at headwater tributaries and is ineffective in reducing the peaking of the natural hydrographs. The fan shaped drainage basin is conducive to a flashy run-off but the relative timing of the Beards Brook and Shedd Brook run-offs determines the width and magnitude of the peak. The stream slopes and areal distribution are shown on Plate I-3.

d. Meteorological and Stream Flow Data.- (1) Climate.- A general summary of the climate prevailing over the Contoocook River watershed can be best described by reference to Page 5, Chapter 1 of Hydrometeorological Report No. 1 "Maximum Possible Precipitation Ompompanoosuc Basin", which states:

"The entire region is so situated geographically that it receives a maximum frequency of visitation of cyclonic storm activity through the entire year, in fact, this area eventually comes under the influence of the majority of cyclonic disturbances which affect the United States. No seasonable variation in precipitation is important, annual rainfall being evenly distributed throughout the months. In general, short period rainfall intensities are greatest in late spring and summer and excessive rains of longer durations occur in late summer and early fall. The extremes of average annual precipitation in New England range from 35 inches in northern Vermont and New Hampshire, with the exception of Mt. Washington where it is considerably higher due to orographic effects in the White Mountains, to 47 inches in southwestern Connecticut and 48 inches on the central coast to Maine. Excessive 24-hour amounts during winter months, December to March, of more than three inches are rare, but summer rainfall excesses are more frequent because of high intensities experienced in thunderstorms. A very even distribution of thunderstorm occurrence makes all parts of the region liable to high short-period rainfall intensities during the summer months.

"Since great accumulations of snow may occur and since the types of cyclonic disturbances which visit this region may result in rapid melting, snow is an extremely important factor in flood production. At Boston, Massachusetts, the average annual snowfall is 43.8 inches, with extremes of 96.4 inches during the winter of 1873-74 and 10 inches during the winter of 1875-76. The average seasonal snowfall at first order Weather Bureau stations ranges from 50.4 inches at Albany, New York, 60.4 inches at Portland, Maine, and 73.0 inches at Concord, New Hampshire, to 38.6 inches at New Haven, Connecticut, and 32.4 inches at Providence, Rhode Island. The greatest average seasonal snowfall from any reporting station is 168.3 inches at Pittsburgh, New Hampshire."

(2) Precipitation Records.- Precipitation records are available at two stations: Jackman Power Plant, Hillsboro, New Hampshire, and Bradford, New Hampshire. The Jackman gage has been a recording gage since 1939 and is considered the best obtainable source of data. Records from adjacent localities were analyzed and used for verifying storm data for run-off studies. Inasmuch as the Jackman Station is within the basin and has a Thiessen's weighting of 75% of the total, it was decided to use this station alone in computing rainfall run-off studies for the basin, except for the analysis of the September 1938 storm. For general information, Table I lists selected rainfall stations showing normal monthly and average annual precipitation. In addition, the normal monthly and average temperatures for these stations, where available, are listed in Table II. A complete table of comparative data and extremes covering the climatology of the U. S. Weather Bureau of Concord, New Hampshire, is listed in Table III to supplement the information contained in Tables I and II.

TABLE I. PRECIPITATION - INCHES

Station Years of Record	Fitzwilliam 21 years Average Monthly	Franklin 39 years Average Monthly	Keene 49 years Average Monthly	Manchester 66 years Average Monthly	Nashua 57 years Average Monthly	Newport 12 years Average Monthly
January	3.21	3.03	2.89	3.29	3.43	3.26
February	2.64	2.70	2.68	2.96	3.32	2.26
March	3.63	3.33	3.21	3.61	3.70	3.65
April	3.71	3.53	3.09	3.19	3.32	3.77
May	3.30	3.19	3.11	3.10	3.00	3.01
June	4.36	3.73	3.23	3.20	3.13	3.63
July	4.24	3.77	3.80	3.43	3.38	3.64
August	3.97	3.47	3.86	3.39	3.53	3.37
September	4.15	3.99	3.60	3.38	3.42	3.78
October	3.20	2.88	2.76	3.02	3.00	2.57
November	3.88	3.25	3.03	3.33	3.25	3.01
December	3.13	3.05	2.99	3.25	3.34	2.76
Average Annual	43.42	39.92	38.25	39.15	39.82	38.71

TABLE II. TEMPERATURE - FAHRENHEIT

Station Years of Record	Franklin 40 years Average Monthly Normal	Keene 48 years Average Monthly Normal	Manchester 12 years Average Monthly Normal	Nashua 15 years Average Monthly Normal
January	19.9	21.1	24.1	24.2
February	20.0	21.2	23.9	24.4
March	30.9	32.0	32.9	33.3
April	42.9	43.7	43.6	43.8
May	55.1	55.3	56.4	56.3
June	63.9	63.6	65.8	65.2
July	69.5	68.8	70.3	70.2
August	66.6	66.3	68.8	68.2
September	59.4	59.4	60.5	60.8
October	48.1	48.6	49.0	49.8
November	35.9	36.7	38.9	39.1
December	23.9	24.8	27.4	27.3
Annual Normal	44.7	45.1	46.6	46.9

TABLE III.

CLIMATOLOGICAL DATA FROM OBSERVATIONS AT U. S. WEATHER BUREAU, CONCORD, N. H.

	TEMPERATURE					PRECIPITATION IN INCHES		UNMELTED SNOWFALL IN INCHES	
				Extremes					
Month	Mean Max.	Mean Min.	Mean Monthly	Highest	Lowest	Monthly Mean	Greatest in 24 hrs.	Monthly Mean	Greatest in 24 hrs.
Length of Record Years	74	74	74	74	74	91	74	74	42
Jan.	29.2	8.9	19.0	72	-35	2.97	2.10	17.6	19.0
Feb.	31.0	9.6	20.3	68	-37	2.65	2.06	17.3	15.0
Mar.	38.4	19.8	29.1	82	-16	3.15	2.59	11.9	12.9
Apr.	52.5	30.6	41.6	92	7	2.96	2.37	4.6	18.3
May	64.8	40.9	52.9	98	22	3.07	3.05	0.1	2.5
June	73.5	49.9	61.7	101	32	3.23	4.47	T	T
July	79.0	55.9	67.4	102	38	3.70	5.11	0	0
Aug.	76.4	54.6	65.5	99	33	3.50	3.32	0	0
Sept.	69.0	47.1	58.0	96	20	3.48	5.97	T	T
Oct.	58.5	37.1	47.8	92	16	3.26	3.45	0.1	1.0
Nov.	44.4	26.4	35.4	80	-17	3.29	4.04	5.4	13.3
Dec.	32.9	15.2	24.0	65	-24	2.92	2.43	11.9	9.5
Year	54.2	33.0	43.6	102	-37	38.18	5.97	68.9	19.0

(3) <u>Stream Flow Data</u>.- The U. S. Geological Survey maintained a gaging station on North Branch of the Contoocook River from 1924 to present, which has a drainage area of 54.8 square miles. The maximum peak discharge during this period was 4680 c.f.s., recorded during the March 1936 flood. Peak discharge values determined by the U. S. Geological Survey for a Bristol gage recorder on Beards Brook near the mouth, from December 1943 to the present show a maximum discharge of 3300 c.f.s. on 24 June 1944.

Discharge records for other drainage areas nearby or adjacent were studied to obtain run-off characteristics. Records for the following stations were used in this study.

Basin	Name of Stream	Station Location	Drainage Area Sq.Mi.	Maximum Discharge c.f.s./ Sq.Mi.	Date of Record	Length of Record
Merrimack	Contoocook R.	East Jaffrey	36.1	99	Sept. 1938	Flood Observation*
Merrimack	Nubanusit Brook	W.Peterboro	45.2	92	Sept. 1938	Flood Observation*
Merrimack	N.Branch Contoocook R.	Antrim	54.8	85	Mar. 1936	21 years
Merrimack	Warner River	Bradford	19.7	120	Sept. 1938	Flood Observation*
Connecticut	S.Br. Ashuelot River	Webb	36.0	165	Sept. 1938	25 years
Connecticut	Otter Brook	Keene	42.3	145	Sept. 1938	22 years

* Observations available only for September 1938 flood.

A hydrograph for the period 1924 to 1942 has been constructed for general information, and is shown on Plates I-4 and I-5. This hydrograph is plotted directly from U. S. Geological Survey records for the North Branch of Contoocook River at the station mentioned above.

The following table illustrates the magnitude of the discharge from the daily average to the spillway design flood. All figures quoted are for the U.S.G.S. Gaging Station on the North Branch at Antrim (Drainage area 54.8 sq.mi.) with the exception of the spillway design flood which is the computed flow at the dam site (Drainage area 64 sq.mi.).

Average flow	115 c.f.s.
Average annual flood	1,650 c.f.s.
Flood of November 1927	2,100 c.f.s.
Flood of March 1936	4,680 c.f.s.
Flood of Sept. 1938	3,850 c.f.s.
Spillway Design Flood (Reservoir Inflow)	23,900 c.f.s.

e. Determination of Spillway Capacity.- (1) General Criteria.- The spillway capacity for Beards Brook is determined by procedures set up in Engineer Bulletin R&H No. 9, 1938. The spillway design flood is based on the highest possible rainfall and very high run-off factors and is assumed to occur when all outlets are inoperative and the reservoir is filled to spillway lip. All storage affecting the spillway design flood is surcharge storage based on these assumptions. The elevation of the top of the dam is determined by adding a minimum freeboard of 5 feet to the maximum water surface elevation in the reservoir produced by the spillway design flood modified by Jackman Reservoir failure. The analytical investigations made for the selection of the spillway design flood and spillway capacity are described in the succeeding paragraphs.

(2) Maximum Storm Studies.- The summer-fall storm as determined from previous studies produces the maximum flood condition on this size drainage area. Detailed analyses were made of the summer-fall limiting storm conditions to be expected over the watershed. The rainfall intensity curve for summer-fall conditions shown on Plate I-14 is based on the maximum possible rainfall for the Contoocook River Basin as determined by the Hydro-meteorological Section of the U. S. Weather Bureau. The flood-producing storm is of 24-hour duration with a total rainfall of 18.38 inches. The spillway design flood for the Beards Brook Reservoir is computed in two separate sections due to the difference in the run-off characteristics of the North Branch section and Beards Brook section of the drainage area and it was necessary to construct two concurrent incremental pluviographs which combined represent the maximum storm conditions over the total drainage area. The rainfall over the total area was determined to be 18.38 inches from the rainfall curves shwon on Plate I-14. Using an infiltration factor of 0.05"/hour determined from previous rainfall run-off studies as a reasonable value for extreme flood conditions, the total run-off was determined to be 56,150 d.s.f. The storm was then centered over

the Beards Brook section of the drainage area as it has the most rapid run-off characteristics and the greatest effect on the spillway floods. The rainfall distribution for 56 square miles was interpolated from the curves of Plate I-14 at 19.58 inches. Allowing for infiltration at the rate of 0.05"/hour, the total run-off over the Beards Brook section was determined to be 28,012 day-sec-ft. The difference in volume between the total for 120 square miles and the incremental run-off wascomputed to be the run-off volume of the North Branch section. This run-off was distributed in proportion to the Beards Brook run-off.

(3) Unit Hydrographs.- (a) North Branch.- It was necessary to compute unit hydrographs for both the North Branch section and the Beards Brook section of the total drainage area. Sufficient reliable stream gaging data is available from the U.S.G.S. station on the North Branch near Antrim (54.8 sq.mi.) to provide a basis for unit graph determination on the North Branch section. The automatic recording rain gage station at the Jackman Hydro-Electric Plant of the New Hampshire Public Service Company in Hillsboro is practically in the basin, and since its Thiessen's weighting is approximately 75% of the total, unit hydrograph studies were based on this station alone. The September 1938 and the June 1944 storms were analyzed for unit graphs and the results shown on Plates No. I-6, I-7, I-8, and I-9. The greater of these unit graphs (the Sept. 1938) was adjusted for drainage area correction, and formed the basis for unit hydrograph No. 1. (Plate I-12). The other two computed graphs were peaked and the peak lag shortened to obtain unit graphs believed more representative of what could be expected during rains of the extreme severity of the spillway design flood. These adjusted graphs are also shown on Plate No. I-12.

(b) Beards Brook.- Less data is available for unit graph studies on the Beards Brook section but for the last two years a pressure-recording gage has been maintained on Beards Brook about 50 feet above its confluence with the North Branch. Because of mechanical difficulties with this type of gage and the uncertainty of the adequacy of the control, a reliable rating curve has never been made for this station. A tentative rating curve has been established by the U.S.G.S. based on two current meter measurements and one slope area determination. A unit graph was computed using the gage data for the hydrograph of September 1944 and is shown on Plates I-10 and I-11. The Jackman rainfall station was used as in the case of the North Branch studies. Inasmuch as this unit graph is only applicable to storms of moderate rainfall its use was discounted in the construction of the computed spillway floods. Three unit graphs were constructed by assuming peak values and peak lag

TABLE 4

MERRIMACK RIVER BASIN

COMPUTED UNIT HYDROGRAPH COEFFICIENTS

Location	Tributary	D.A.	Date of Storm	Excess Rainfall Inches	Flood Peak	C.F.S./Sq. Mi.	q_p	L	L_{ca}	t_p	t_r	t_p/t_r	C_t	$640C_p$
Lincoln	Pemigewasset	104	June 1942	.57	3,580	34.4	56.1	12.8	7.0	6.1	3	2.0	1.59	342
"	"	104	June 1942	.57	3,580	34.4	67.3	12.8	7.0	5.6	1	5.6	1.42	378
Woodstock	"	193	June 1942	1.00	10,900	56.5	64.2	17.2	11.0	7.9	1	7.9	1.64	508
"	"	193	June 1942	1.00	10,900	56.5	59.5	17.2	11.0	8.0	3	2.7	1.66	476
Plymouth	"	622	June 1942	1.78	25,500	41.0	24.1	38.1	15.7	15	3	5.0	2.21	362
Frank. Falls	"	1,000	Nov. 1927	3.44	69,000	69.0	20.0	63.6	32.0	19	6	3.2	1.93	380
" "	"	1,000	Sept. 1938	4.19	57,300	57.3	17.6	63.6	32.0	28	6	4.7	2.84	493
Wentworth	Bakers	58.8	June 1942	3.74	11,800	201	57.9	12.4	6.6	7	1	7.0	1.87	405
"	"	58.8	June 1942	3.74	11,800	201	55.1	12.4	6.6	7	3	2.3	1.87	385
Rumney	"	143.0	June 1942	3.15	21,400	150	51.8	19.8	8.8	6.5	1	6.5	1.38	337
"	"	143.0	June 1942	3.15	21,400	150	49.0	19.8	8.8	7.2	3	2.4	1.53	318
Keene	Otter Brook	41.4	Sept. 1938	6.00	6,130	148	37.1	10.0	6.3	6.5	3	2.2	1.88	241
Webb	S.B.Ashuelot	36.6	Sept. 1938	6.00	5,960	163	41.9	7.2	3.9	6.5	3	2.2	2.39	273
Bennington	Contoocook	186	Sept. 1938	7.40	15,400	82.8	12.2	25	13.8	27	3	9	4.70	329
"	"	186	June 1944	2.38	5,100	27.4	10.3	25	13.8	27	3	9	4.70	278
"(Inflow)	"	186	Sept. 1938	7.40	18,000	96.9	16.1	15.4	7.7	13	3	4.3	3.10	209
Blackwater	Blackwater	128	Sept. 1938	4.64	6,880	53.8	16.6	21.3	15	27	6	4.5	4.78	448
Antrim	N.B.Contooc.	54.8	Sept. 1938	5.95	3,850	70.2	13.3	16.1	11.1	23.5	3	7.8	4.97	314
"	" " "	54.8	June 1944	2.27	1,300	23.7	9.5	16.1	11.1	46.5	3	15.5	9.81	442
Hillsboro	Beards Brook	64	Sept. 1944	1.03	1,200	18.8	22.3	10.9	6.1	15	3	5	4.25	269

TABLE 5
CONTOOCOOK RIVER BASIN
COMPARISON OF SNYDER'S UNIT GRAPH COEFFICIENTS FOR RESERVOIR SPILLWAY STUDIES

RESERVOIR	L	L_{ca}	q_p Unit Graph #1	q_p Unit Graph #2	q_p Unit Graph #3	T_p Unit Graph #1	T_p Unit Graph #2	T_p Unit Graph #3	C_t Unit Graph #1	C_t Unit Graph #2	C_t Unit Graph #3	640 C_p Unit Graph #1	640 C_p Unit Graph #2	640 C_p Unit Graph #3
Mountain Brook (DA = 14 S.M.)	5.4	3.85	80	≠110	150	5.0	≠ 4.5	4.0	2.0	≠1.8	1.6	400	≠496	598
Beards Brook (DA = 56 S.M.)	10.9	6.13	30	50	≠ 70	8	7	≠6	2.27	≠1.99	≠1.71	240	349	≠420
West Peterboro (DA = 44 S.M.)	15.7	6.20	34.6	≠ 50.8	69.8	7.4	≠ 6.3	5.5	1.87	≠1.6	1.4	256	≠320	386
Bennington (DA = 168 S.M.)	15.4	7.70	16.1	≠ 28.2	35.6	13	≠11	9	3.10	≠2.62	2.14	209	≠256	320
North Branch (DA = 64 S.M.)	27.5	14.55	13.3	20	≠ 25	23.5	18	≠15	4.40	3.36	≠2.80	314	360	≠374
Blackwater (DA = 128 S.M.)	21.3	15.0	16.6*	27.3		27*	16		4.78*		2.83	448*	436	

Tabulation arranged in order of magnitude of L_{ca}
*Based on 1938 Flood
≠ Adopted for Spillway Design Flood

values based on computed coefficients for other locations in the Merrimack River basin. (Table 4.) A separate study was made of the relative timing of the North Branch and Beards Brook hydrograph peaks for all storms on which data were available. This study indicated that the peak lag (time interval between the center of gravity of the pluviograph and the peak of the hydrograph) was approximately 17 hours for Beards Brook and approximately 45 hours for the North Branch. The comparison of the unit graphs adopted for computing the spillway floods on Beards Brook is shown on Plate I-13.

(c) Results of Unit Hydrograph Studies.- Table 4 contains a summary of the coefficients for unit hydrographs derived from a study of flood hydrographs for various gage sites in the Merrimack River Basin and adjoining tributaries. An explanation of the meaning and derivation of these coefficients is contained in the article, "Synthetic Unit Graphs", by F. F. Snyder, in the "Transactions of the Geophysical Union" for 1938. The floods studied are all comparatively recent in order to take advantage of the records of new stream-gaging stations and recording rainfall stations. The drainage areas for the floods analyzed vary considerably in run-off and topographic characteristics as may be noted by the wide divergence in the value of the coefficients.

(d) Comparison of Unit Hydrograph Coefficients. - Table 5 tabulates the pertinent unit hydrograph data used for deriving the spillway floods for all of the reservoirs in the Contoocook River Basin. The values are somewhat higher than those shown on Table 4 derived from storms and floods of record but the values are more applicable for the high rates of rainfall used in the spillway floods. The wide divergence in the unit graph coefficients between the North Branch and Beards Brook is due primarily to the differences in the drainage basin characteristics, and secondly to the fact that the run-off data on the North Branch for the past 20 years substantiates the sluggish nature of this drainage basin. The lack of data on Beards Brook, and a study of the shape and nature of the drainage basin makes it expedient to assume the higher conservative values.

(4) Routing Criteria for Spillway Floods.- The general criteria established for routing the spillway design flood through the reservoir are as follows: At the beginning of the selected spillway design flood the reservoir is filled to the crest of the spillway (Elevation 705 M.S.L.) and all outlets, gated and ungated are inoperative. Allowance is not made initially for friction or head losses in the diversion channel

that tend to make the water surface elevation in the Beards Brook section of the reservoir slightly higher than the water surface in the North Branch section. This is a conservative method of routing as it takes no account of the incremental storage in Beards Brook caused by the head loss in the diversion channel. Actually this incremental storage would tend to reduce the spillway discharge. The reservoir outflow is computed for a free overfall ogee spillway using the weir formula $Q = CLH^{3/2}$ where "L" = 450 feet and values of "C" of an ogee section are used from 3.0 to 3.8 for the maximum head (See Plate III-5). A detailed analysis of the spillway is included in Appendix III. Since the unit hydrograph is synthetic, it is considered that it applies to reservoir inflow, consequently valley storage is neglected and the spillway floods are routed through the reservoir using the gross storage above the spillway crest.

(5) Computed Spillway Floods.- Plate 15 shows the six computed spillway floods derived from the maximum possible precipitation of the summer-fall flood (See Par. C-2) combined with the six empirical unit hydrographs shown on Plates I-12 and I-13. The three unit hydrographs, of different characteristics on both sections of the drainage area, are used to obtain a range in computed spillway floods, in order to determine the effect on the design criteria of the spillway. Unit hydrograph No. 1 is considered applicable to all periods of rainfall less than two inches in a three-hour period, unit hydrograph No. 2 is used for rainfall excess values exceeding two inches, and graph No. 3 is used for similar rainfall excess periods. Routing these three combined hydrographs through the reservoir, with the spillway length of 450 feet, results in maximum spillway surcharges that vary between 7.8 and 11.0 feet. (See summary and graph on Plate I-16). This large difference in surcharge head indicates that with a constant volume of inflow, the spillway discharge changes materially with variation in the shape and intensity of the hydrograph. To determine the effect of variations in volume, the ordinates of hydrograph "A" were increased 25%, 50% and 100%, and the resulting hydrographs were routed through the surcharge storage. The hydrographs are not illustrated but the effect of the increased volume is shown on Plate I-16, by the curve of maximum water surface elevation versus the percent increase in volume. There is no great difference between the curves of constant volume and increasing volume, principally because the limited amount of surcharge storage makes the peak inflow the governing criterion.

(6) Spillway Design Flood.- (a) Selection.- Hydrograph "C" is selected as the spillway design flood after consideration of the various factors involved in the method of analyzing precipitation, run-off, and constructing flood hydrographs, their relative degree of accuracy, and their effect on the spillway requirements. The individual and combined hydrographs with the spillway discharge, reservoir stage elevation, and pluviograph, are summarized on Plate I-17. Pertinent data relative to the spillway design flood are tabulated below.

Rainfall in inches in 24 hours	18.38
Rate of infiltration (inches per hour)	0.05
Run-off in inches, North Branch	16.35
Run-off in inches, Beards Brook	18.60
Run-off volume in d.s.f., North Branch	28,140
Run-off volume in d.s.f., Beards Brook	28,010
Peak inflow in c.f.s., North Branch	23,900
Peak inflow in c.f.s., Beards Brook	57,000
Peak inflow in c.f.s., combined	66,500
Peak spillway discharge in c.f.s	62,500
Maximum water surface (ft. above M.S.L.)	716.0
Surcharge storage utilized, acre feet	10,100
Surcharge storage utilized, inches	1.6

(b) Myers Coefficient.- The Myers Coefficient for the composite inflow peak of 66,500 c.f.s. used in the expression $Q = C\sqrt{\text{Drainage Area}}$, is approximately 6060. The comparable coefficients for the incremental peaks is 2990 for the North Branch and 7600 for Beards Brook. It should be noted that the peak inflow and Myers coefficient for the North Branch is based on the assumed distribution of rainfall in which the maximum intensities fall on the Beards Brook drainage area. Paragraph f of this appendix shows the construction of a hydrograph in which it is considered that the maximum rainfall intensities fall on the drainage area of the North Branch. This condition gives a peak flow on the North Branch of 27,000 c.f.s. (Plate I-23) with a corresponding Myers Coefficient of 3380.

(c) Discussion.- The selection of hydrograph "C" as the Spillway Design Flood was predicated principally on the relation of the maximum water surface elevation to the peak inflow as plotted on Plate I-16. The results of routing the spillway floods indicate that the reservoir surcharge storage is so small that there is no appreciable reduction in the peak flow, and the spillway discharge is nearly equal to the reservoir inflow. In view of the uncertainties in constructing the unit hydrograph for Beards Brook and the sensitivity of the reservoir in respect to the limited surcharge storage it is believed advisable to select the higher flood for design purposes. However, it should be noted also that the maximum water surface elevation in the reservoir from routing hydrograph "C" is only 1.3 feet higher than the water surface obtained with hydrograph "B". The net result, therefore, of adopting hydrograph "C" instead of "B" is an increase of approximately one foot in the height of the dam.

(d) Valley Storage.- The valley storage obviously is negligible on the North Branch because of the limited extent of that section of the reservoir. An estimate of the valley storage on the Beards Brook section of the reservoir has been made by backwater computations. Because of the limited area of the reservoir the storage during the spillway design flood is only about 2000 acre-feet. Consequently, as the spillway floods are based on a theoretical flood-producing storm and synthetic unit hydrograph, the effect of valley storage is neglected and the spillway flood is considered an inflow to the reservoir and the gross surcharge storage is considered effective in the routing computations.

(e) Effect of Jackman Dam Failure.- 1. Description of Jackman Dam.- In the event of a flood of the proportions of the Spillway Design Flood on the North Branch of the Contoocook River the earth dam of the New Hampshire Public Service Company at Jackman Reservoir would be over-topped and would fail. This dam is located on the North Branch about 1/2 mile above the proposed flood control dam site. The spillway has a net length of 106 feet and is designed for a maximum discharge of 10,200 c.f.s. with a 3-foot freeboard. It is theoretically capable of discharging approximately 15,000 c.f.s. without wave ride-up.

2. Failure Analysis.- Numerous studies were made of the different types and degrees of failure that could occur and of the effect on the spillway requirements for Beards Brook. Results of these studies indicate that the factors involved in the failure are of a very indeterminate nature. It is impossible, (a) to predict how much over-topping the dam could withstand safely, (b) to determine the location of the weakest section of the dam, or (c) to compute the length of time it would take to completely erode a gap in the dam once failure started. For the purpose of this analysis a section of the embankment four hundred feet in length adjacent to the concrete overflow section was established as the section that would fail instantaneously down to the original earth line, as soon as the dam was overtopped. The plan of failure is shown on Plate I-19. This is considered to be the maximum breach that could occur in the short time the dam would be over-topped since if this section were breached, it would quickly relieve the pressure on the rest

of the dam.

3. Hydrograph of Failure.- An approximate storage capacity curve was constructed for the Jackman Reservoir and also a rating curve for the spillway with all flashboards washed out. A rating curve for the breached section also was computed with critical flow conditions at the failed section. These curves are shown on Plate I-20. The Spillway Design Flood for the North Branch was then routed through Jackman Reservoir with the failure occurring when the reservoir stage reached the top of the dam. This analysis resulted in an instantaneous maximum discharge from Jackman Reservoir through the assumed breach of 80,000 c.f.s. with a total drawdown of the Jackman pool of 4,400 acre feet. It is impossible to draw down more than 4,400 acre feet of the Jackman Pool because of the necessity of maintaining sufficient head on the breached opening to maintain the combined discharge of the Spillway Design Flood plus the storage depletion discharge. Failure of the embankment was estimated to occur when Jackman Spillway Discharge reached 15,000 c.f.s. The hydrograph of the failure is graphically illustrated on Plate I-20. However, in order to allow for any small discrepancies in the relative peak timing of the Spillway Design Floods on the North Branch Section to the Beards Brook Section, caused by errors in the unit graphs, the hydrograph of the dam failure was synchronized with the maximum reservoir stage attained in the Beards Brook Reservoir during the routing of the Spillway Design Flood. This hydrograph was then combined with the Beards Brook Spillway Design Flood inflow hydrograph and routed through the total surcharge storage at the Beards Brook Reservoir. The water surface reached a maximum stage elevation of 717.9, or 1.9 feet higher than obtained by the Spillway Design Flood. The combined flood inflow was 127,000 c.f.s. and the maximum spillway discharge was 79,300 c.f.s. The combination routing is shown on Plate No. 21.

f. Freeboard.- The theoretical freeboard required for wave height is computed by the methods prescribed in Engineer Bulletin, R.&H. No. 9, 1938. The data used in these computations are tabulated below.

F = Fetch in miles	= 1.9
V = Wind velocity, miles per hr.	= 80
A = Angle of wind and fetch	= 0
D = Depth of water in feet	= 75

Using the formula of the Lorentz Zuderzee Commission of Holland, the allowance for wind setup, S, is computed as follows:

$$S = .00125 \frac{V^2F}{D} \cos A = 0.20 \text{ feet}$$

Using the Stevenson-Molitor formula the allowance for wave height, h, is computed as follows:

$$h = 0.17\sqrt{VF} + 2.5 - \sqrt[4]{F} = 3.43 \text{ feet}$$

The allowance for ride-up of waves on the sloped surface of the dam was taken as 1.4 times the total height of wave, 3.43 feet as computed above, and equals 4.80 ft. The freeboard required is the summation of wind set-up and wave ride-up and equals 5.0 feet. Since the computed freeboard is equal to the minimum value of 5 feet prescribed in Engineer Bulletin R.&H. No. 9, 1938, this value is used for determining the elevation of the top of dam.

g. Determination of Top of Dam.- The elevation of the top of the dam embankment is determined by adding the required freeboard to the maximum water surface elevation in the Beards Brook section of the reservoir at the time of the maximum head on the spillway. Friction losses due to flow through the canal from Beards Brook to the North Branch are computed to be 0.9 feet which is added also to the maximum water surface elevation in the North Branch section during the spillway design flood. The top elevation of the dam embankment is determined by the following data:

Elevation, crest of spillway	705.0
Maximum head on spillway from spillway design flood modified by failure of Jackman Dam	12.9
Allowance for canal losses	0.9
Freeboard	5.0
	723.8
Adopted elevation for top of dam	723.0

It was decided to select the height of the dam to the nearest lower foot inasmuch as it is considered improbable that all the adverse criteria will occur concurrently and that it is therefore reasonable and justifiable to encroach 0.8 of a foot on the computed elevation. The criteria used for establishing the height of the dam are summarized briefly and are as follows:

1. Reservoir stage at spillway crest at beginning of flood.
2. All outlets inoperative
3. Storm centered over Beards Brook drainage area to produce maximum flood.
4. Jackman Dam to fail instantaneously and to occur concurrently with maximum reservoir stage in the Beards Brook reservoir.
5. Hurricane wind to occur during period of maximum reservoir stage.

It is to be noted that with this selected elevation the net effect of the failure of the Jackman Dam is one additional foot on the height of the dam.

h. Canal Flow Studies.-Plates I-22 and I-23 show hydrographs of canal discharges for various flood conditions. Plate I-22 indicates (1) the canal flow with the failure of Jackman Dam occurring concurrently with the maximum reservoir stage, and (2) the canal flow during the spillway design flood without failure of Jackman Dam. The canal flow during the spillway design flood is entirely in the direction from the Beards Brook section of the reservoir toward the North Branch section except for the brief surge caused by the instantaneous failure of the Jackman Dam. The maximum canal flow in both cases is approximately 51,000 c.f.s. with an average velocity of 7.4 feet per second. The maximum reverse flow is approximately 17,400 c.f.s. Plate I-23 shows the results of a special study to determine the conditions producing the maximum velocities of flow in the canal. The flood flows are based on spillway flood criteria, that is, the storm is derived from the limiting rainfall curves (Plate I-14) with the same unit hydrographs used in computing the spillway design flood (Plate I-17). However, in this analysis the storm is centered over the North Branch drainage area, and the reservoir is considered to be empty at the beginning of the storm. This combination of conditions creates the maximum flood discharge on the North Branch occurring when the reservoir stages are so low that flow through the canal will be unaffected by backwater. The hydrograph shows that the maximum flow through the diversion channel from the North Branch to Beards Brook is 10,000 c.f.s., after which the direction of flow reverses and the flow from Beards Brook to the North Branch reaches a peak of 27,500 c.f.s. Plate I-23 also shows the relation of the reservoir elevation in the Boards Brook section with respect to the magnitude and direction of flow in the canal. It should be noted that the most adverse velocities in the channel will occur with the water flowing from the North Branch to Beards Brook despite the fact that the rate of flow is lower than the rate of flow in the reverse direction. This feature results from the comparative depths of flow, as the flow from the North Branch will occur at a time when the reservoir stage in the Beards Brook section is too low to still the channel discharges. Additional hydraulic discussion relative to the channel flow conditions is contained in Appendix III.

i. Reservoir Design Flood.- (1) Reservoir Capacity.- Results of studies for flood control reservoirs in this district indicate that, if possible, it is desirable to provide approximately 6 inches of storage to most effectively control the maximum flood of record without exceeding the downstream channel capacity. Six inches of storage at Beards Brook would require 38,800 acre feet with the spillway crest at elevation 709.5. The resulting cost of the dam at this elevation would be prohibitive and elevation 705 was selected as spillway lip for economical reasons. The unit cost per acre foot increases rapidly when the spillway lip is raised above elevation 705. This is the result of three separate factors, namely: the encroachment on property valuations in Hillsboro Lower Village above the dam site, the necessity for additional protection to the Jackman hydro-electric plants' penstock, and the disproportional increase in the construction cost of the dam section. This elevation provides a storage capacity for Beards Brook of 35,000 acre feet or 5.5 inches over the drainage basin. This capacity provides effective flood control for all floods up to and including the March 1936 which was the most severe flood of record from a reservoir operating standpoint.

(2) Downstream Channel Capacity.- Field investigation indicates that the channel capacity of the North Branch is approximately 2,500 c.f.s. in the main stream below the confluence with Beards Brook. However, no material damage is caused by overbank flows up to approximately 3,000 c.f.s. Route 202 Highway Bridge can safely pass approximately 3,500 c.f.s. without damage to the structure. This was shown during the freshet of June 1944 which had a peak discharge of 3300 c.f.s. measured at the Bristol Gage upstream. A discharge of 3,500 c.f.s. will not seriously affect the tailwater of the Jackman Hydro-electric plant. However, to definitely eliminate over-bank flow on the Contoocook River in conjunction with the operation of the proposed Bennington Reservoir it is planned to limit the total discharge from the reservoir to flows between 2500 and 3000 c.f.s.

(3) Normal Operation.- During periods of normal flow, the spillage from the Jackman Reservoir on the North Branch will be discharged through the 6' x 4' ungated outlet in the spillway. In the Beards Brook conduit, it is proposed to leave the 7' x 4' gate open to discharge the normal flow of the river. Ordinarily there will be no storage retained in either section of the reservoir, nor will there be any flow in the diversion canal.

(4) Operation During Floods.-The size of outlets and gates have been designed to simplify reservoir operation. It is proposed to leave only the 7' x 4' gate open during the

period of a flood in addition to the spillway ungated outlet. Consequently, there will be no difference between the normal operation and the operation during a flood, and the reservoir will function as a simple retarding basin with fixed outlets. The only operation of the gates will occur after a flood in order to expedite emptying the reservoir. The layout of the structures and characteristics of the reservoir is conducive to fixed outlet operation because in both the North Branch and Beards Brook sections of the reservoir considerable head and discharge capacity develops on the outlets without utilizing reservoir storage. The discharge capacity of the spillway outlet is approximately 500 c.f.s. with the water surface at elevation 690, the crest elevation of the weir in the diversion canal. If the flood is of such magnitude to exceed this rate of discharge, the surplus flood waters will be diverted over the canal weir into the Beards Brook section of the reservoir. The height of water in the reservoir is dependent on the volume and rapidity of the storm run-off. As the flood flow recedes on the Contoocook River following the flood, the Beards Brook Reservoir will be emptied by increasing and maintaining the discharge at the safe channel capacity. The gates will be operated as indicated on Plate I-26.

(5) Effect on 1936 and 1938 Floods.- The two maximum floods of record, occurring in March 1936 and September 1938, have been used as reservoir design floods to check the adequacy of the storage, the design discharge, the proposed method of operation, and the probability of spillway discharges during the operation scheme. These two floods are selected to illustrate the range in the reservoir's effectiveness; the 1936 being a prolonged three-storm flood with considerable volume, and the 1938 a single peak flood caused by one concentrated storm producing practically the same peak magnitude as the 1936 flood. The hydrographs of the two floods are based on computed peak discharges for Beards Brook with the shape and volume of the hydrographs determined from flood records on adjacent and comparable rivers, and the observed hydrograph at the U.S.G.S. gage on the North Branch near Antrim, N.H. adjusted for drainage area. The North Branch hydrographs, based on the gaging-station, are inflow hydrographs due to the location of the gage. The constructed hydrographs for Beards Brook represent the reservoir inflow and consequently the composite hydrographs were routed through the gross reservoir storage. Plates I-24 and I-25 show the effect of the reservoir on the 1936 and 1938 floods utilizing the reservoir initially as a simple retarding basin, gate operation being required only during the period of receding stages in the reservoir, to permit faster emptying of the reservoir, within the limits of safe channel capacity. Only three steps in the gate operation

are illustrated, but it is possible to have the same four steps shown on Plate I-26. Data pertinent to these studies are summarized as follows:

	1936 Flood	1938 Flood
Combined peak inflow, c.f.s.	10,200	10,000
North Branch peak inflow, c.f.s.	5,400	4,400
Beards Brook peak inflow, c.f.s.	6,300	6,000
Combined Discharge at maximum reservoir stage c.f.s.	2,710	2,300
North Branch outlet Discharge at maximum reservoir stage c.f.s.	860	590
Beards Brook outlet Discharge at maximum reservoir stage c.f.s.	1,850	1,710
Maximum Reservoir Stage	705.0	692.7
Flood Control Storage Ac. Ft.	35,000	25,800

j. Time to Empty Reservoir.- Assuming the reservoir at spillway crest, elevation 705.0 with a constant inflow of 5 c.f.s. per sq. mile, and a gate operation as shown on Plate I-26 it will take about 10 days to completely empty the reservoir. At the end of four days using schedule A (one 7'x4' gate open) 2.5 inches of storage, equivalent to 45% of the total storage is available. Three days later, using operating schedule B (2 - 7' x 3' gates open) about 4.4 inches of storage is available equivalent to 80% of the total capacity. The available storage at the end of seven days is considered adequate protection against a possible second flood. The time and discharge relationship for emptying the reservoir are shown also on Plates I-24 and I-25 for the March 1936 flood and the September 1938 flood.

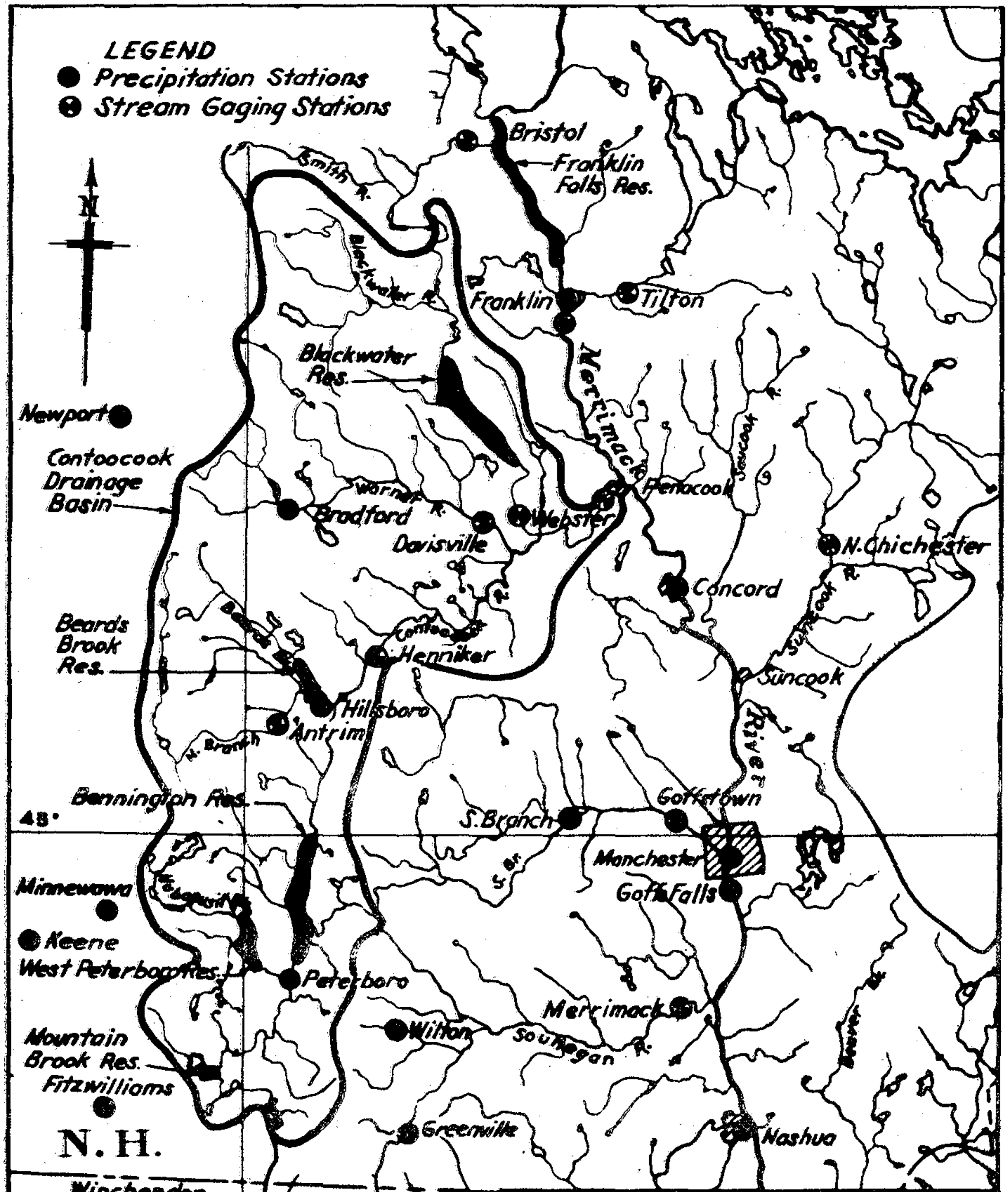
k. Cofferdam Design.- The construction schedule requires the building of the conduit through the earth section of the Beards Brook dam in the dry, offset from, and approximately parallel to, the natural brook channel. During this phase of construction, the natural channel will handle all anticipated flows. After the conduit is constructed it is planned to cofferdam the natural stream bed and divert the brook through the conduit during construction of the closure section of the dam. All available flow records on Beards Brook were studied in computing the height of the cofferdam. The flood selected as a criteria for the cofferdam design was that of June 25, 1944, resulting from a rainfall totalling over 5 inches in 30 hours. The flow on Beards Brook had a discharge peak of 3300 c.f.s. and an estimated net run-off of 2.26" or 3400 acre feet. Routing this hydrograph through the conduit, with all gate passages open resulted in a pool elevation of 644.4. Due to the severity of the storm causing this flood, it was decided to set yhe elevation of the top of the cofferdam at 645 without providing additional height for freeboard.

1. Stage Frequency Curve.- A study was made to determine the probable reservoir stage equalled or exceeded in a designated number of years. Two distinct steps were required; (1) to construct a natural discharge volume frequency curve for the Beards Brook Reservoir site, and (2) to construct a stage frequency curve for the reservoir which is a direct result of the inflow volume. Available stream flow data for North Branch and Beards Brook, required for the first step is limited to 20 years of record on the North Branch near Antrim, New Hampshire, (Drainage Area 54.8 square miles) from August 1924 to present, and practically no records for Beards Brook. Because of limited essential stream flow data at the reservoir site it was necessary to utilize the records of an adjacent watershed for comparison. Otter Brook near Keene, New Hampshire, (Drainage Area, 41 square miles) with records available from October 1923 up to the present time was selected for this purpose. Comparison of the North Branch records with the concurrent Otter Brook records indicates that differences of considerable magnitude frequently occur in comparable one day volumes, however, comparable maximum two-day volumes per square mile from the two drainage areas check very closely. Consequently it was assumed that Beards Brook (Drainage Area 56 square miles) would have a comparable two-day volume per square mile. It was also determined that the reservoir stage is dependent principally on volume, and that the type of hydrograph, that is a flash flood or a long flat flood, has little influence on the reservoir stage. It was, therefore, concluded that stream flow records in North Branch and Otter Brook, modified for the Beards Brook Reservoir drainage area could be utilized. For these data a frequency curve of two-day volumes was constructed using the formula, $Y = \frac{M}{N-0.5}$, in which

Y = the probability of occurrence in years
M = the years of available records, and
N = the summation of occurrences

Several floods of computed frequency were then routed through the reservoir to obtain the maximum reservoir stage with the prescribed method of gate operation. The results of these computations are plotted graphically on Plate I-25. Listed below are the anticipated pool elevations for floods of the tabulated frequency.

Frequency	Reservoir Pool Elevation
1	644.0
2	650.4
5	661.0
10	670.0
20	680.0
50	694.0
100	705.0

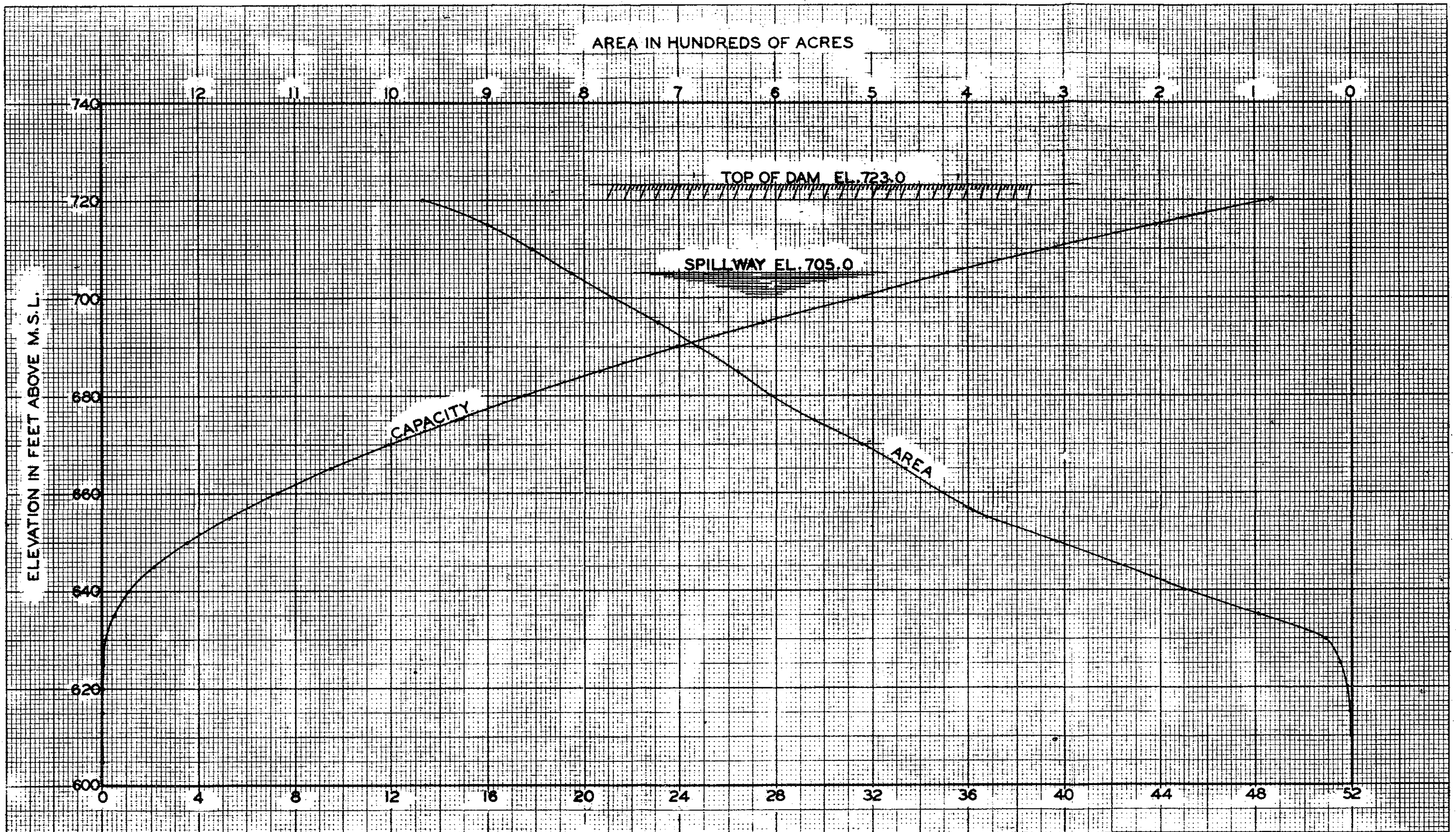


MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK RIVER BASIN

PRECIPITATION AND STREAM GAGING STATIONS

U.S. ENGINEER OFFICE BOSTON, MASS.
FILE NO. M 100-50/4 18 APRIL 1945

PLATE I-1

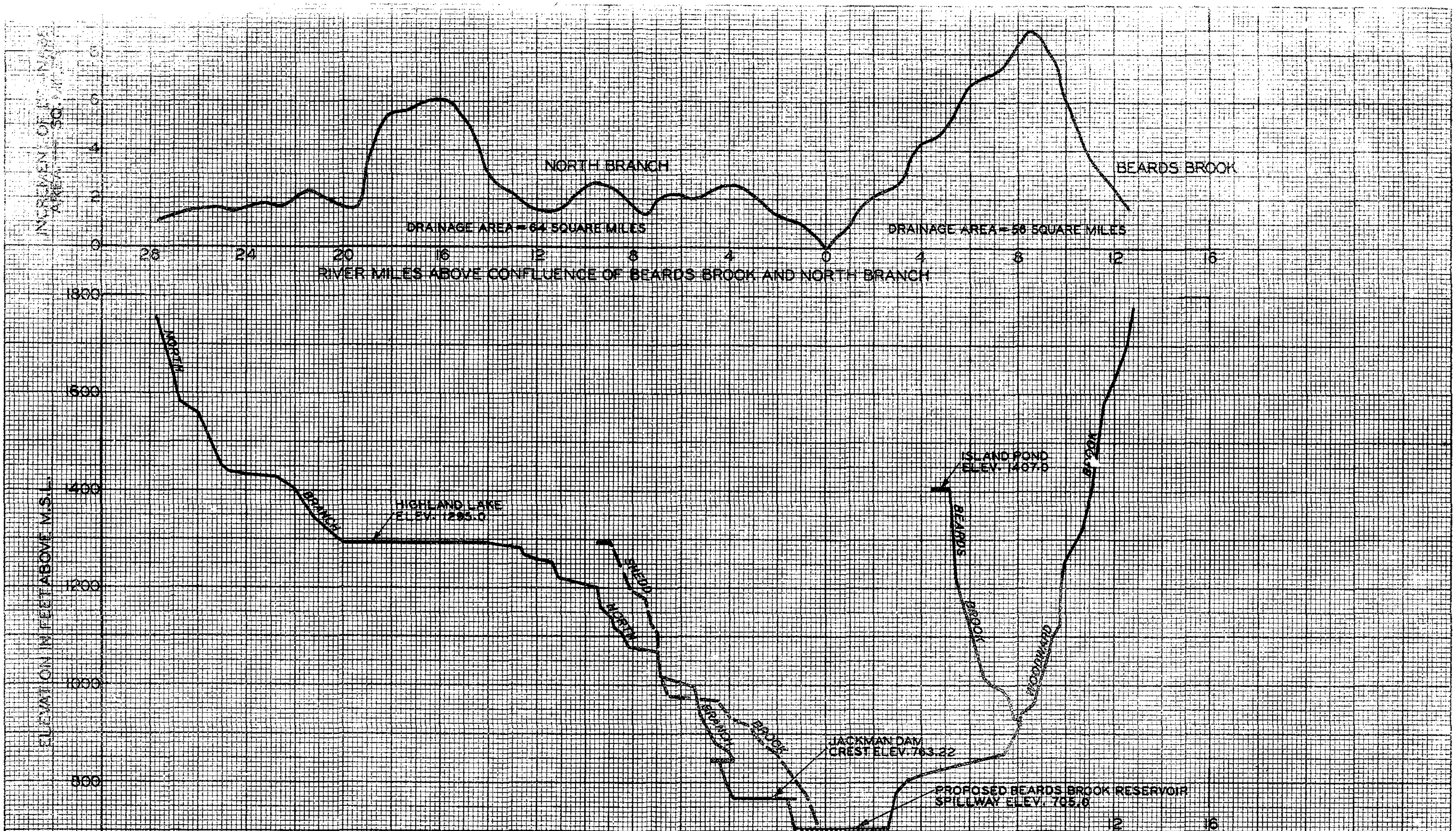


MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK AND NORTH BRANCH

AREA AND CAPACITY CURVES

U. S. ENGINEER OFFICE BOSTON, MASS.
FILE NO. M 16-12/28

PLATE I-2



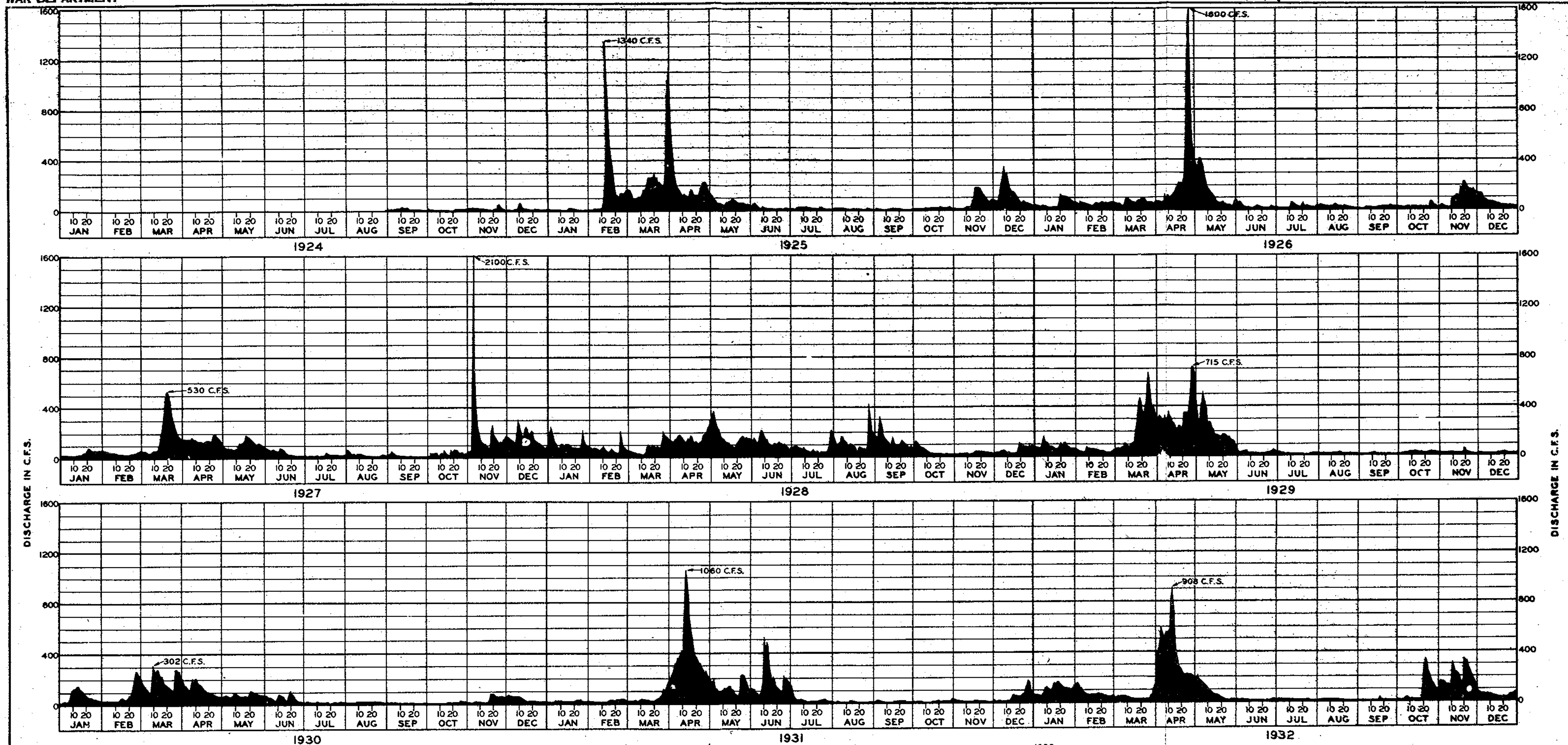
MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

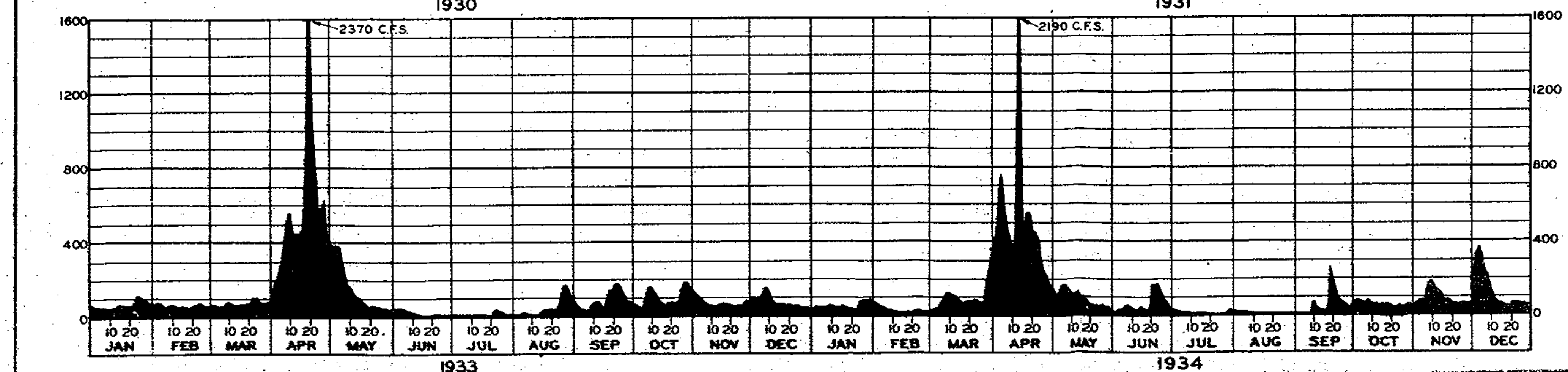
DRAINAGE BASIN CHARACTERISTICS

U.S. ENGINEER OFFICE. BOSTON, MASS.
FILE NO. M16-12/29

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY





Note:
Discharge figures indicated represent maximum instantaneous flows for the water year. Other ordinates represent mean daily discharges.
All data is obtained from U.S.G.S. station near Antrim N.H.
(Drainage area = 54.8 sq. miles)

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
HYDROGRAPH
OF NORTH BRANCH
1924–1934

U.S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

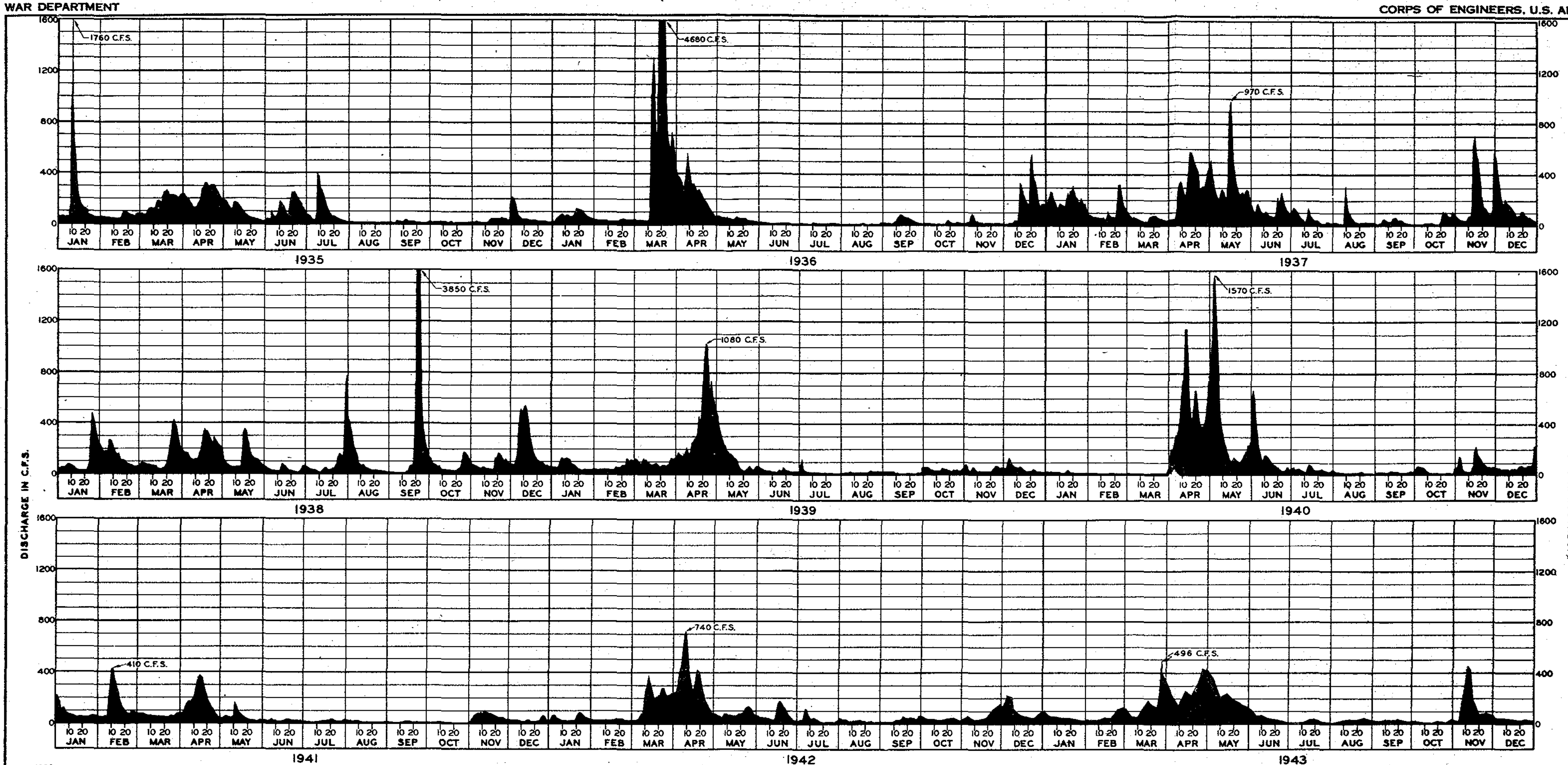
APPROVED: [signature] COLONEL, CORPS OF ENGINEERS, DIVISION ENGINEER

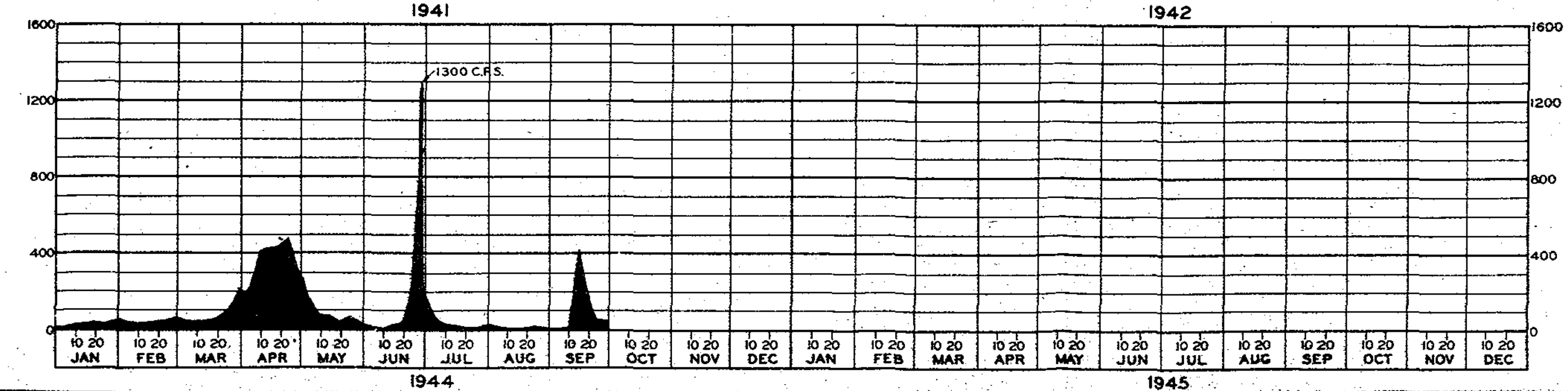
SUBMITTED: R.P. Brown, CHIEF DESIGN BRANCH

Elliot F. Chase

FILE NO. M 16-12/30

PLATE I-





Note:
Discharge figures indicated represent maximum instantaneous flows for the water year. Other ordinates represent mean daily discharges.
All data is obtained from U.S.G.S. station near Antrim N.H.
(Drainage area = 54.8 sq. miles)

NO.	CHARACTER	DATE	APPR

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
HYDROGRAPH
OF NORTH BRANCH
1935 – 1942

U.S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945

APPROVAL RECOMMENDED:

APPROVED:

SUBMITTED:

FILE NO. M16-12/31

PLATE 1-

Computed by EL.P.
Date 1 May 1945

UNIT HYDROGRAPH DETERMINATION

STREAM North Branch LOCATION Near Antrim, N.H.

DRAINAGE AREA 54.8 SQ. MI.

STORM OF September 1938 PREPARED BY Boston DIST. New Eng. DIV

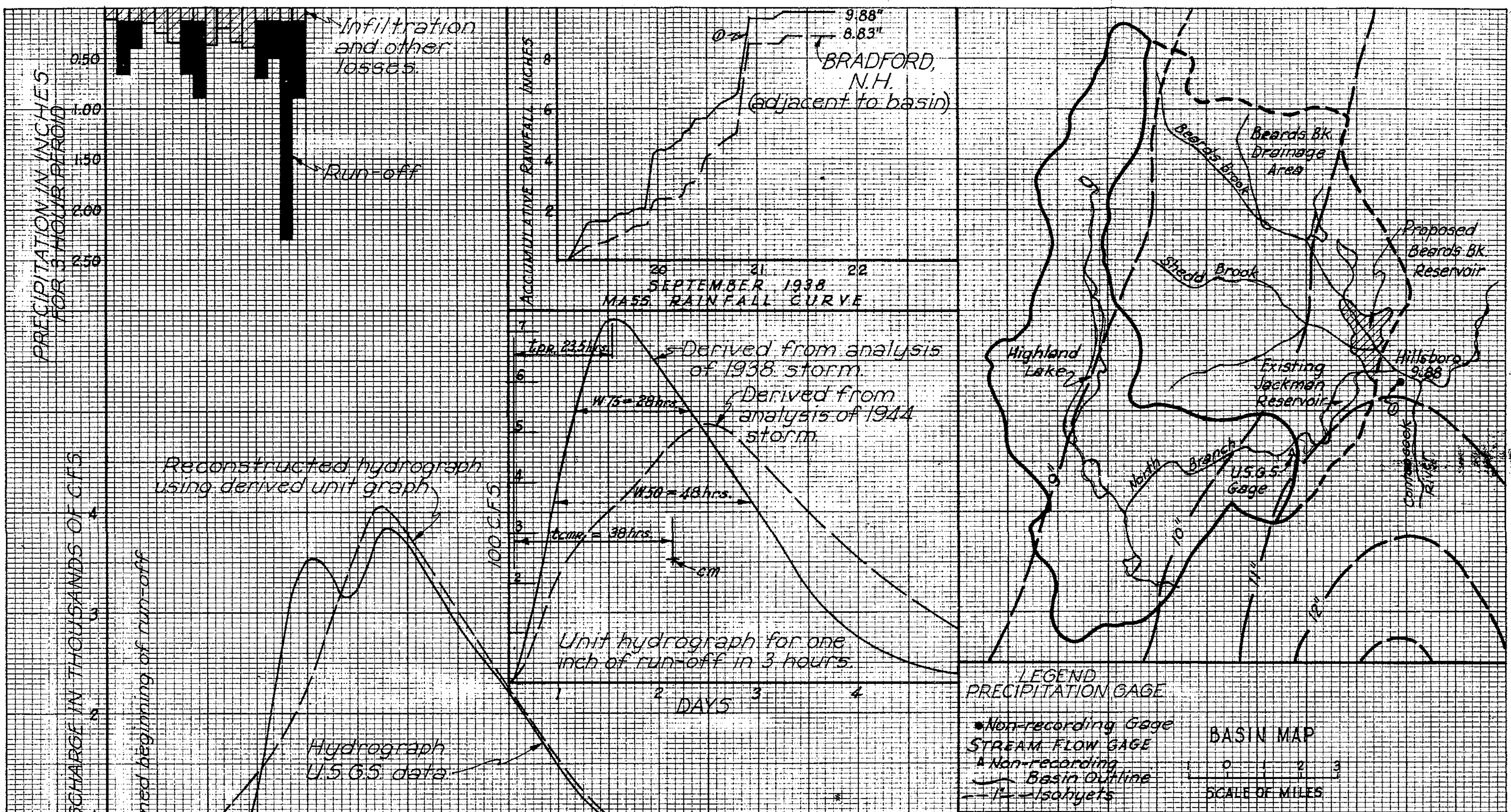
AV. RAINFALL 9.61 IN.; RAINFALL-EXCESS 5.95 IN.; F_{av}, 0.061 IN/HR.

L 24.0 mi, L_{ca} 11.05 mi; $(LL_{ca})^{0.3}$ 5.35 ; t_R 3 hrs;

LAG(t_{pR}) 23.5 hrs; C_{tR} 4.40 ; q_{pR} 13.3 cfs./sq. mi; C_p640 314 ;

LAG(t_{cmR}) 38 hrs; W_{50} 48 hrs; W_{75} 28 hrs; SLOPE

TIME	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	OBSERVED 3 HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
3						
6	125	100	25	60		145
9	160	107	53	150		200
M	205	114	91	270		275
3	250	121	129	400		375
6	300	128	172	510		500
9	350	135	215	600		650
20-N	400	142	258	675		815
3	475	149	326	720		975
6	590	156	434	718		1120
9	800	163	637	697		1275
M	1250	170	1080	665		1420
3	1950	177	1773	630		1560
6	2650	184	2466	595		1760
9	3275	191	3084	560		2000
21-N	3520	198	3322	525		2250
3	3530	205	3325	490		2625
6	3340	212	3128	455		2975
9	3150	219	2931	417		3325
M	3250	226	3024	380		3645
3	3550	233	3317	340		3950
6	3820	240	3580	300		4050
9	3800	247	3553	260		3950
22-N	3670	254	3416	218		3800
3	3500	261	3239	183		3600
6	3300	268	3032	155		3420
9	3070	275	2795	130		3225
M	2860	282	2578	108		3000
3	2670	289	2381	90		2825
6	2520	296	2224	75		2625
9	2370	303	2067	70		2425
23-N	2200	310	1890	50		2250
3	2010	317	1693	40		2075
6	1850	324	1526	30		1975
9	1650	331	1319	23		1900
M	1460	338	1122	20		1850
3	1300	345	955	18		1825
6	1140	352	788			
9	1025	359	666			
24-N	920	366	554			
3	860	375	485			
6	800	380	420			



CONTOOCOOK RIVER
NORTH BRANCH NEAR ANTRIM N.H.
UNIT HYDROGRAPH DETERMINATION
STORM OF SEPT. 1938

IN SHEETS SHEET NO.

U.S. ENGINEER OFFICE, BOSTON, MASS.

FILE NO. M16-12/33

PLATE I-7

WAR DEPARTMENT CORPS OF ENGINEERS, U. S. ARMY

Computed by E.L.P.
Date 1 May 1945

UNIT HYDROGRAPH DETERMINATION

STREAM North Branch LOCATION Near Antrim, N. H.

DRAINAGE AREA 54.8 SQ. MI.

STORM OF June 1944 PREPARED BY Boston, Mass. DIST. N.E. DIV.

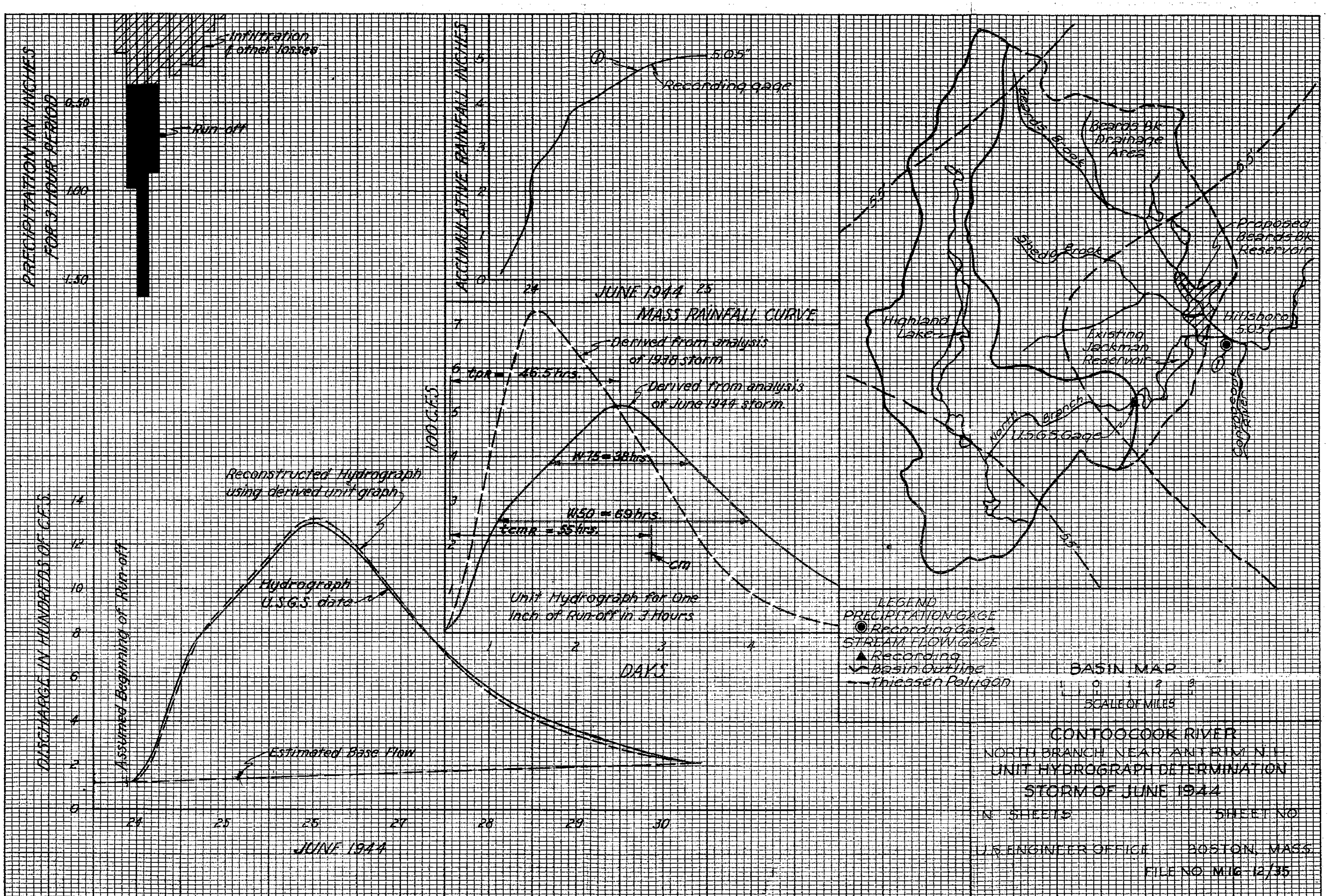
AV. RAINFALL 5.10 IN.; RAINFALL-EXCESS 2.27 IN.; F_{av} 0.105 IN./HR.

L 24.0 mi; L_{ca} 11.05 mi; $(LL_{ca})^{0.3}$ 5.35 ; t_R 3 hrs;

LAG(t_{pR}) 46.5 hrs; C_{tR} 8.69 ; q_{pR} 9.50 c.f.s./sq. mi.; C_p 640 .441

LAG(t_{cmR}) 55.0 hrs; W_{50} 69 hrs; W_{75} 38 hrs; SLOPE

TIME 1944 June	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	OBSERVED HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
9	118	118	0	0		118
24-N	145	120	25	23		130
3	220	122	98	70		200
6	340	123	217	157		300
9	480	125	355	232		460
M	620	127	493	256		590
3	740	129	611	287		720
6	807	131	676	316		807
9	860	133	727	338		880
25-N	907	134	773	367		925
3	960	136	824	393		990
6	1013	138	875	418		1035
9	1070	140	930	445		1090
M	1120	142	978	473		1150
3	1180	144	1036	495		1200
6	1233	145	1088	510		1260
9	1280	147	1133	513		1300
26-N	1300	149	1151	505		1320
3	1280	151	1129	492		1310
6	1247	153	1094	475		1275
9	1210	155	1055	452		1240
M	1160	156	1004	426		1190
3	1115	158	957	402		1130
6	1050	160	890	375		1070
9	995	162	833	343		1000
27-N	930	164	766	317		947
3	875	165	710	287		885
6	820	167	653	267		820
9	770	169	601	242		770
M	720	171	549	222		710
3	670	173	497	202		655
6	630	174	456	182		610
9	590	175	415	168		570
28-N	553	177	376	151		525
3	520	179	341	134		495
6	490	181	309	117		465
9	460	183	277	104		440
M	440	185	255	93		410
3	420	187	233	84		390
6	400	189	211	75		385
9	380	191	189	67		355
29-N	360	193	167	59		337



PRECIPITATION IN INCHES FOR 3 HOUR PERIOD
0.50
1.00
1.50
Infiltration & other losses
Run-off
ACCUMULATIVE RAINFALL INCHES
5.05"
Recording gage
JUNE 1944
MASS RAINFALL CURVE
Derived from analysis of 1938 storm
tpR = 46.5 hrs.
Derived from analysis of June 1944 storm
100 C.F.S.
W75 = 38 hrs.
W50 = 69 hrs.
tcmR = 55 hrs.
cm
Unit Hydrograph for One Inch of Run-off in 3 Hours
DAYS
Reconstructed Hydrograph using derived unit graph
Hydrograph U.S.G.S. data
Assumed Beginning of Run-off
DISCHARGE IN HUNDREDS OF C.F.S.
Estimated Base Flow
JUNE 1944
Beards Bk Drainage Area
Beards Brook
Proposed Beards Bk Reservoir
Shedd Brook
Highland Lake
Existing Jackman Reservoir
Hillsboro 5.05"
North Branch
U.S.G.S Gage
Contoocook River
5.5"
LEGEND
PRECIPITATION GAGE
Recording Gage
STREAM FLOW GAGE
Recording
Basin Outline
Thiessen Polygon
BASIN MAP
SCALE OF MILES
CONTOOCOOK RIVER
NORTH BRANCH NEAR ANTRIM N.H.
UNIT HYDROGRAPH DETERMINATION
STORM OF JUNE 1944
N SHEETS
SHEET NO
U.S. ENGINEER OFFICE BOSTON, MASS.
FILE NO. M16-12/35

PLATE I - 9

Computed by W.M.W.
Date 27 July 1945

UNIT HYDROGRAPH DETERMINATION

STREAM Beards Brook LOCATION Hillsboro, N. H.

DRAINAGE AREA ______ SQ. MI.

STORM OF September 1944 PREPARED BY Boston DIST. New Eng. DIV.

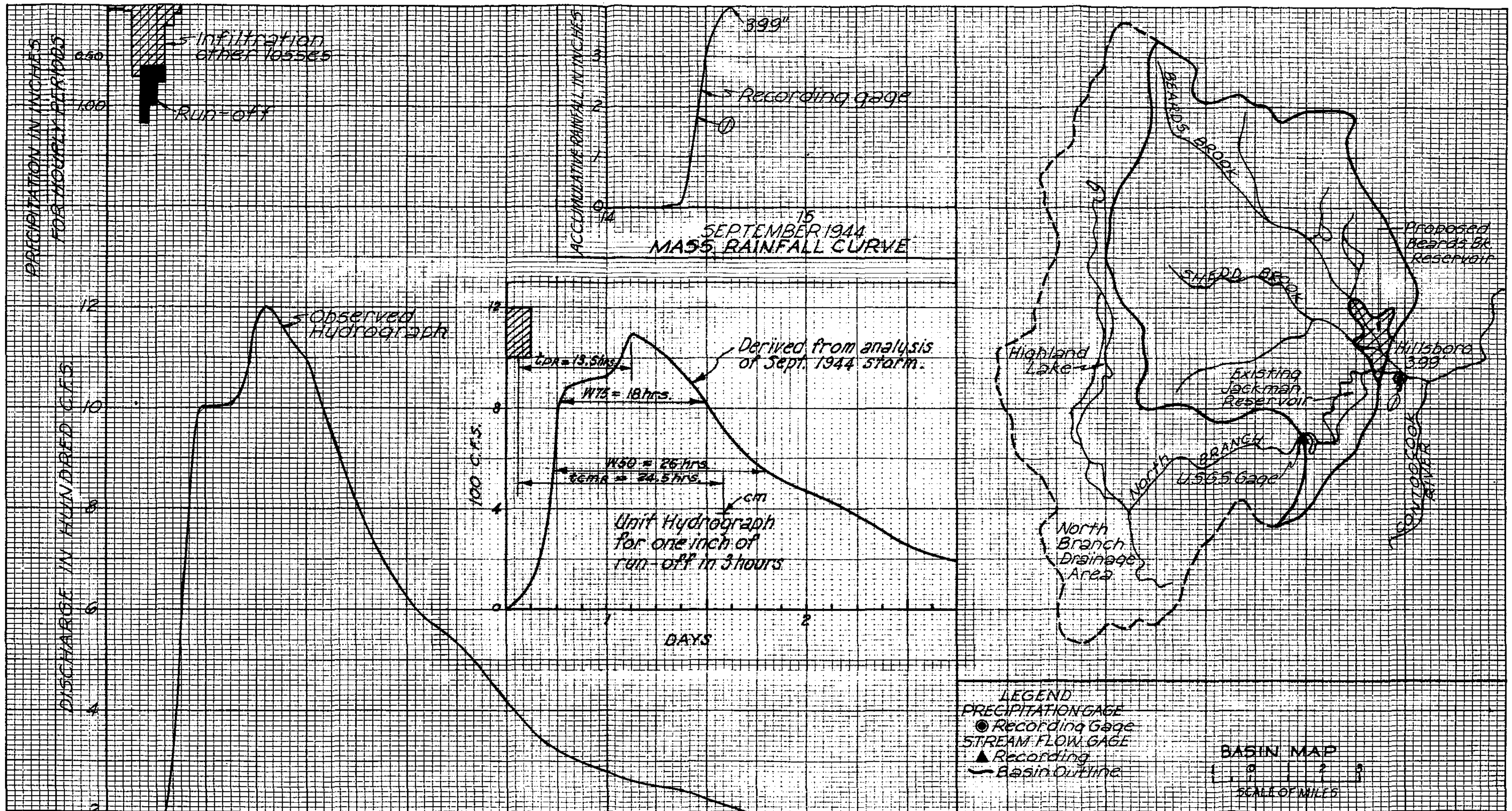
AV. RAINFALL 3.99 IN.; RAINFALL-EXCESS 1.03 IN.; $F_{av.}$ 0.329 IN./HR.

L 10.9 mi; L_{co} 6.13 mi; $(LL_{co})^{0.3}$ 3.52; t_R 3 hrs;

LAG(t_{pR}) 13.5 hrs; C_{tR} 3.82; q_{pR} 19.1 c.f.s./sq.mi.; $C_p 640$ 258

LAG(t_{cmR}) 24.5 hrs; W_{50} 26 hrs; W_{75} 18 hrs; SLOPE ______

TIME	OBSERVED DISCHARGE C.F.S.	ESTIMATED BASE FLOW C.F.S.	STORM RUNOFF C.F.S.	OBSERVED HOUR UNIT HYDROGRAPH C.F.S.	ADJUSTED HOUR UNIT HYDROGRAPH C.F.S.	REPRODUCED STORM HYDROGRAPH C.F.S.
1A	200	70	130		126	130
4	820	71	759		735	759
7	1003	72	931		904	931
10	1030	73	957		929	957
15 13	1200	74	1126		1090	1126
16	1140	75	1065		1035	1065
19	1050	76	974		945	974
22	905	77	828		803	828
1A	780	78	702		680	702
4	675	79	596		579	596
7	605	80	525		510	525
10	560	80	480		466	480
16 13	525	81	444		430	444
16	458	82	376		365	376
19	400	83	317		308	317
22	345	84	261		253	261
1A	312	85	227		220	227
4	290	86	204		198	204
7	272	87	185		180	185
10	260	88	172		167	172
17 13	250	89	161		156	161
16	240	90	150		145	150
19	220	91	129		125	129
22	202	92	110		107	110
1A	188	93	95		92	95
4	175	94	81		79	81
7	165	95	70		68	70
10	160	96	64		62	64
18 13	160	97	63		61	63
16	157	98	59		57	59
19	142	99	43		42	43
22	122	100	22		21	22
1A	112	101	11		11	11
4	110	102	8		8	8
7	103	103	0		0	0
10						
13						
16						
19						
22						



CONTOOCOOK RIVER
BEARDS BROOK
UNIT HYDROGRAPH DETERMINATION
STORM OF SEPT. 1944

IN SHEETS | SHEET NO.

U.S. ENGINEER OFFICE BOSTON, MASS.

FILE NO. M16-12/37

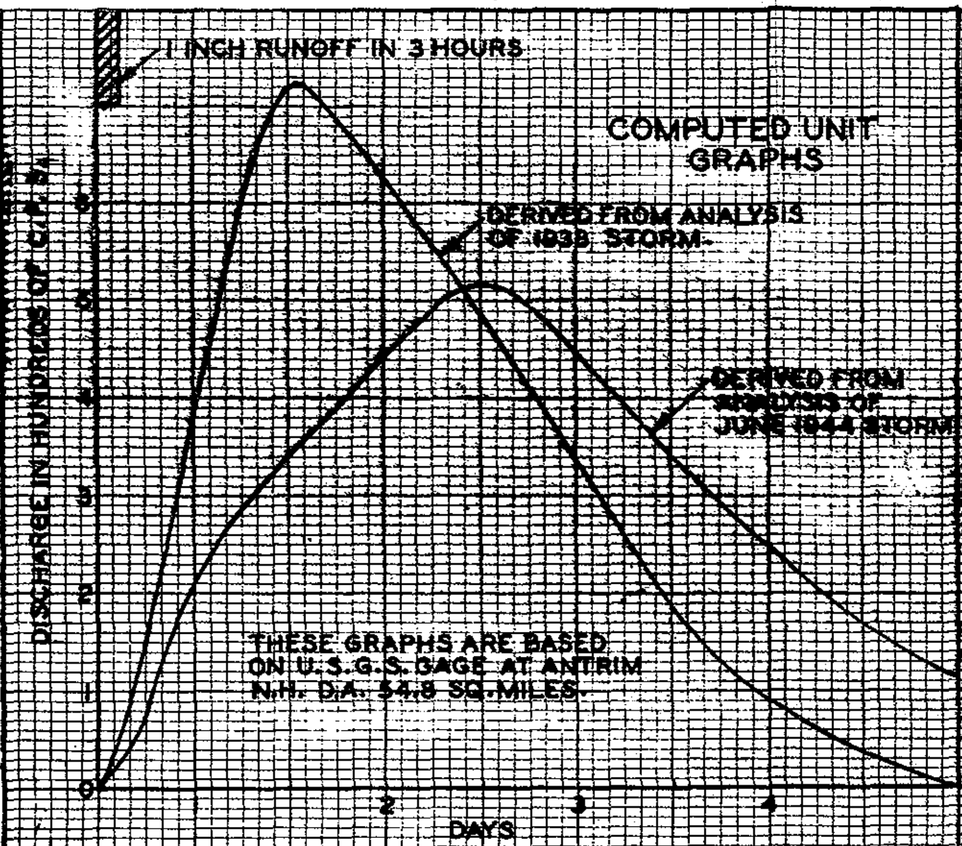
PLATE I-11

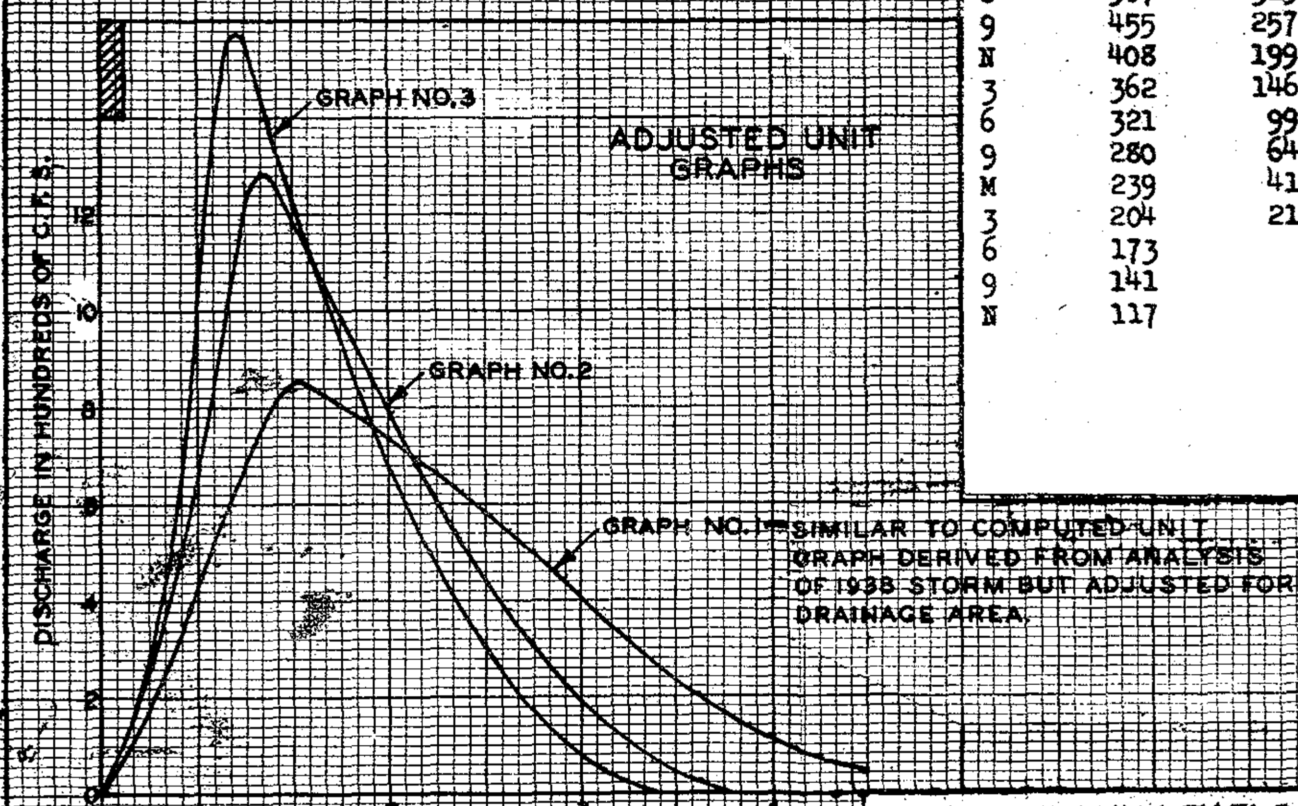
ANALYSIS OF UNIT HYDROGRAPH

ITEM	SYMBOL	EXPLANATION	UNIT	COMPUTED GRAPHS SEPT. '38	JUNE '44	ADJUSTED UNIT GRAPHS #1	#2	#3
(1)	D.A.	Drainage Area	Sq.Mi.	54.8	54.8	64	64	64
(2)		Storm of		Sept. '38	June '44	-	-	-
(3)		Total Weighted Rainfall over Basin	In.	9.61	5.10	-	-	-
(4)		Rainfall Excess	In.	5.95	2.27	-	-	-
(5)	F_{av}	Infiltration	In./Hr.	0.061	0.105			
(6)		Peak Discharge	c.f.s.	3,850	1,300			
(7)		Peak Discharge per Sq. Mi.	c.s.m.	70.4	23.8			
(8)	L	Length of Drainage Basin	Mi.	24.0	24.0	27.5	27.5	27.5
(9)	L_{ca}	Length from C.G. to Point of Discharge	Mi.	11.05	11.05	14.55	14.55	14.55
(10)	$(LL_{ca})^{0.3}$			5.35	5.35	6.04	6.04	6.04
(11)	t_R	Unit Rainfall Duration	Hours	3	3	3	3	3
(12)	Lag(t_{pR})	Time of Peak Concentration	Hours	23.5	46.5	23.5	18.5*	15.5*
(13)	C_{tR}	Drainage Basin Coefficient		4.40	8.69	3.90	3.00	2.60
(14)	q_{pR}	Peak Discharge of Unit Hydrograph	cfs/sq.mi.	13.3	9.50	13.3	20.0*	25.0*
(15)	$C_p 640$	Drainage Basin Coefficient		314	441	310	370	390
(16)	Lag(t_{cmR})	Time of Volume Concentration	Hours	38	55.0	38	28.5	24
(17)	W_{50}	Width at 50% of Peak Discharge	Hours	48	69	47	29	22
(18)	W_{75}	Width at 75% at Peak Discharge	Hours	28	38	28.0	15.3	11.5

*Assumed Values

For detailed explanation of use of above symbols refer to "Synthetic Unit Graphs" by Franklin F. Snyder, Transactions American Geophysical Union, 1938.





NORTH BRANCH OF CONTOOCOOK RIVER

UNIT HYDROGRAPH VALUES

TIME Hour	GRAPH 1 c.f.s.	GRAPH 2 c.f.s.	GRAPH 3 c.f.s.
M			
3	82	105	105
6	181	245	269
9	298	420	537
N	438	642	981
3	580	934	1472
6	695	1226	1576
9	790	1274	1395
M	853	1186	1238
3	833	1074	1098
6	806	988	946
9	770	892	818
N	732	800	689
3	700	707	572
6	666	613	473
9	627	531	385
M	584	455	304
3	545	382	234
6	507	315	169
9	455	257	123
N	408	199	81
3	362	146	50
6	321	99	23
9	280	64	
M	239	41	
3	204	21	
6	173		
9	141		
N	117		

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH
COMPARISON OF UNIT GRAPHS
NORTH BRANCH

U.S. ENGINEER OFFICE — BOSTON, MASS.
OCT. 1945 — FILE NO. M16-12/38

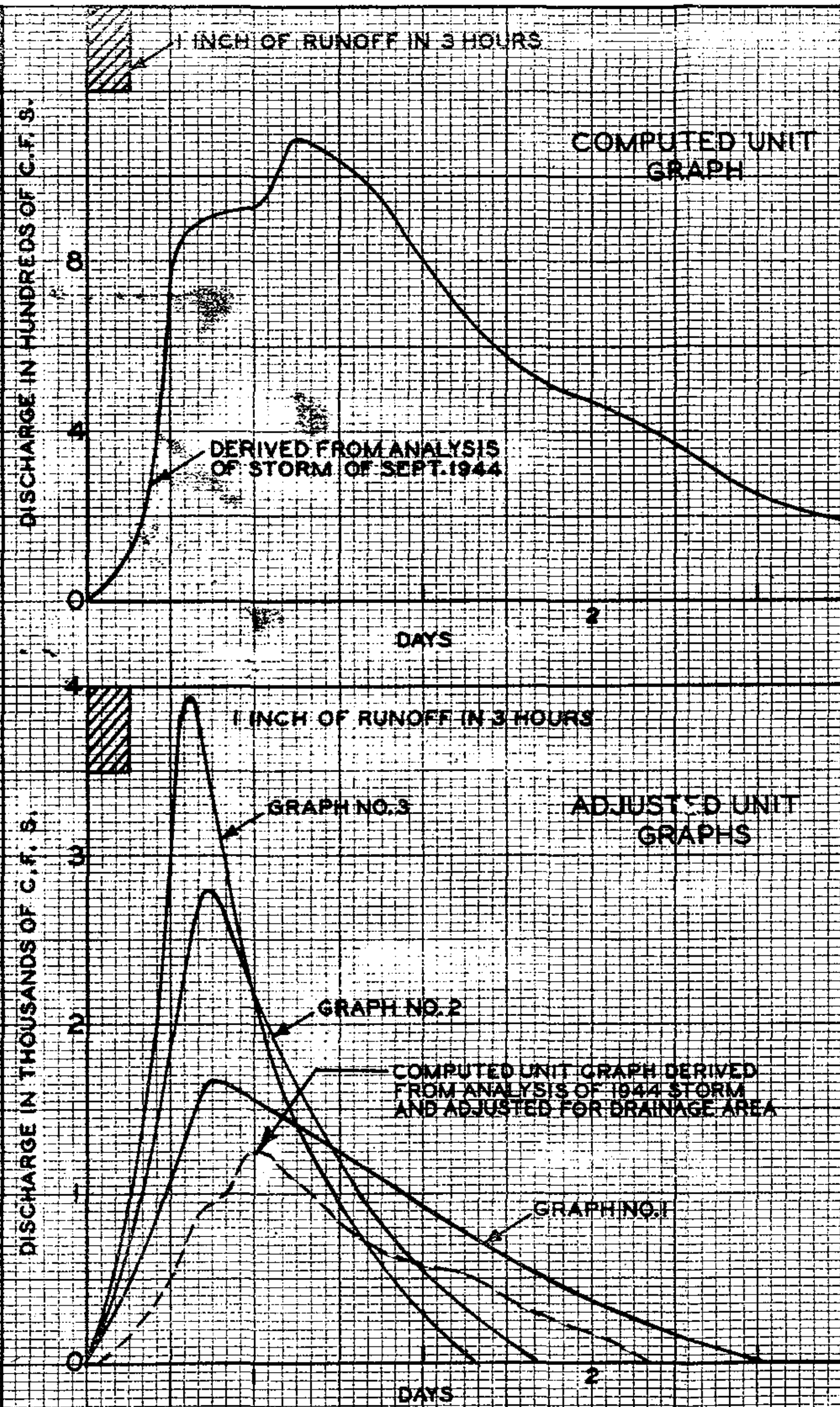
PLATE I-12

ANALYSIS OF UNIT HYDROGRAPH

ITEM	SYMBOL	EXPLANATION	UNIT	COMPUTED GRAPH SEPT. 1944	ADJUSTED UNIT GRAPHS #1	#2	#3
(1)	D.A.	Drainage Area	Sq. Mi.	56	56	56	56
(2)		Storm of		Sept. 1944	-	-	-
(3)		Total Weighted Rainfall over Basin	In.	3.99	- Assumed Unit Graphs	-	-
(4)		Rainfall Excess	In.	1.03	-	-	-
(5)	F_{av}	Infiltration	In./Hr.	0.329	-	-	-
(6)		Peak Discharge	c.f.s.	1,200	-	-	-
(7)		Peak Discharge per Sq. Mi.	c.s.m.	19.1	-	-	-
(8)	L	Length of Drainage Basin	Mi.	10.9	10.9	10.9	10.9
(9)	L_{ca}	Length from CG to Point of Discharge	Mi.	6.13	6.13	6.13	6.13
(10)	$(LL_{ca})^{0.3}$			3.52	3.52	3.52	3.52
(11)	t_R	Unit Rainfall Duration	Hours	3	3	3	3
(12)	Lag(t_{pR})	Time of Peak Concentration	Hours	13.5	7.5	7.0	5.5
(13)	C_{tR}	Drainage Basin Coefficient		3.82	2.10*	2.00*	1.60*
(14)	q_{pR}	Peak Discharge of Unit Hydrograph	cfs/sq.mi.	19.1	29.5	50.5	70.5
(15)	C_p640	Drainage Basin Coefficient		258	222	354	388
(16)	Lag(t_{cmR})	Time of Volume Concentration	Hours	24.5	18	12.2	10.8
(17)	W_{50}	Width at 50% of Peak Discharge	Hours	26	19.8	11.9	8.0
(18)	W_{75}	Width at 75% of Peak Discharge	Hours	18	11.0	6.0	4.0

*Assumed Values

For detailed explanation of use of above symbols refer to "Synthetic Unit Graphs" by Franklin F. Snyder, Transactions American Geophysical Union, 1938.



BEARDS BROOK

UNIT HYDROGRAPH VALUES

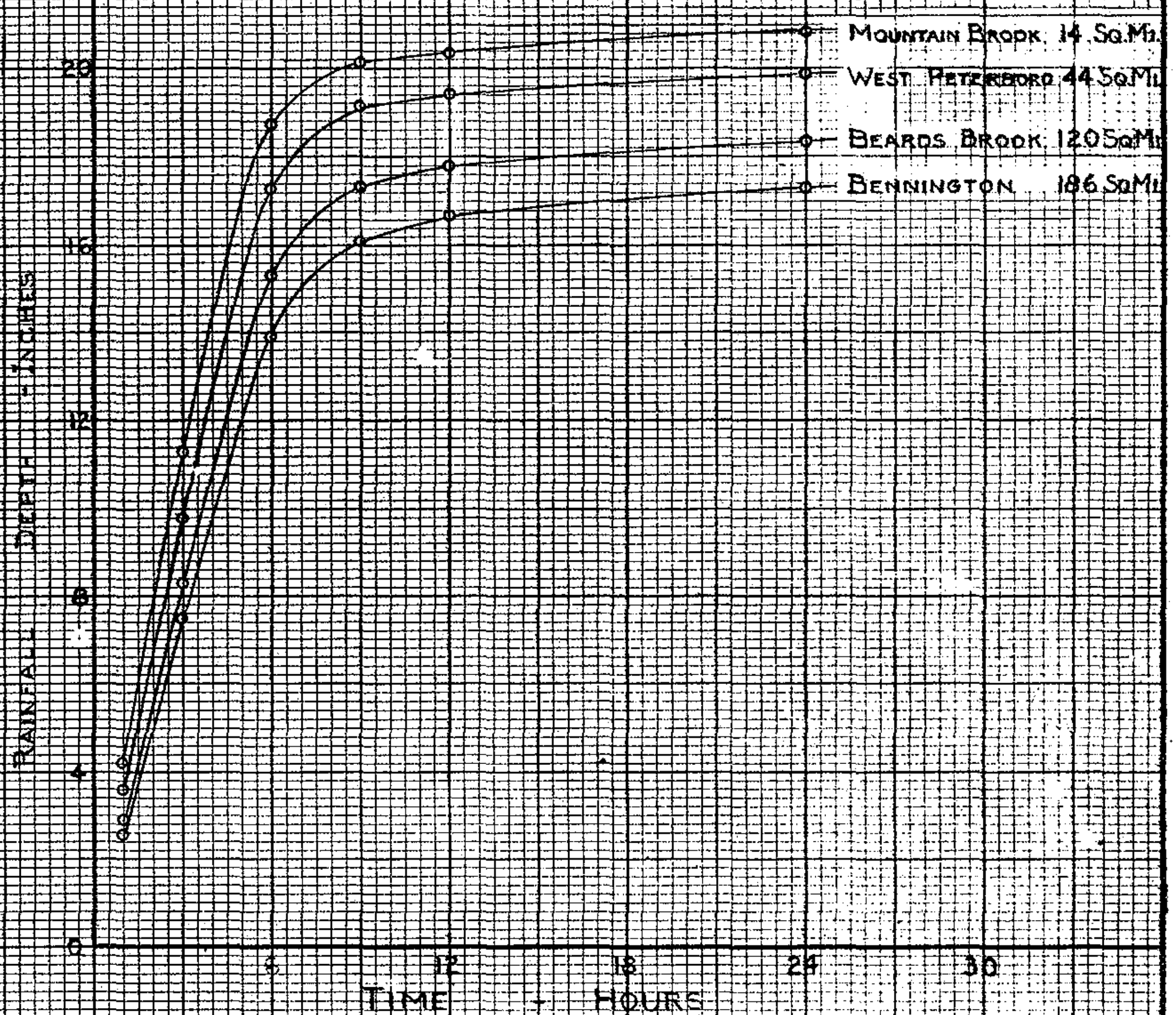
TIME Hour	GRAPH 1 c.f.s.	GRAPH 2 c.f.s.	GRAPH 3 c.f.s.
M			
1	150	210	275
2	325	450	600
3	500	740	950
4	670	1050	1450
5	900	1410	2050
6	1130	1830	2900
7	1350	2300	3850
8	1560	2710	3850
9	1670	2770	3300
10	1660	2600	2900
11	1625	2380	2500
N	1560	2180	2150
1	1510	1980	1850
2	1460	1800	1600
3	1400	1630	1370
4	1350	1475	1220
5	1290	1325	1070
6	1240	1170	920
7	1180	1060	810
8	1140	950	700
9	1070	840	600
10	1025	750	500
11	975	660	400
M	920	560	310
1	875	500	230
2	825	410	160
3	780	340	70
4	730	260	0
5	680	200	
6	640	150	
7	590	80	
8	550	25	
9	500	0	
10	452		
11	425		
N	370		

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

COMPARISON OF UNIT GRAPHS BEARDS BROOK

U.S. ENGINEER OFFICE — BOSTON, MASS.
FILE NO. M16-12/39

THESE CURVES ARE BASED ON DATA SUPPLIED BY THE HYDROMETEOROLOGICAL SECTION OF THE WEATHER BUREAU. REFER TO LETTER FROM CHIEF OF ENGINEERS, 8 DECEMBER 1944

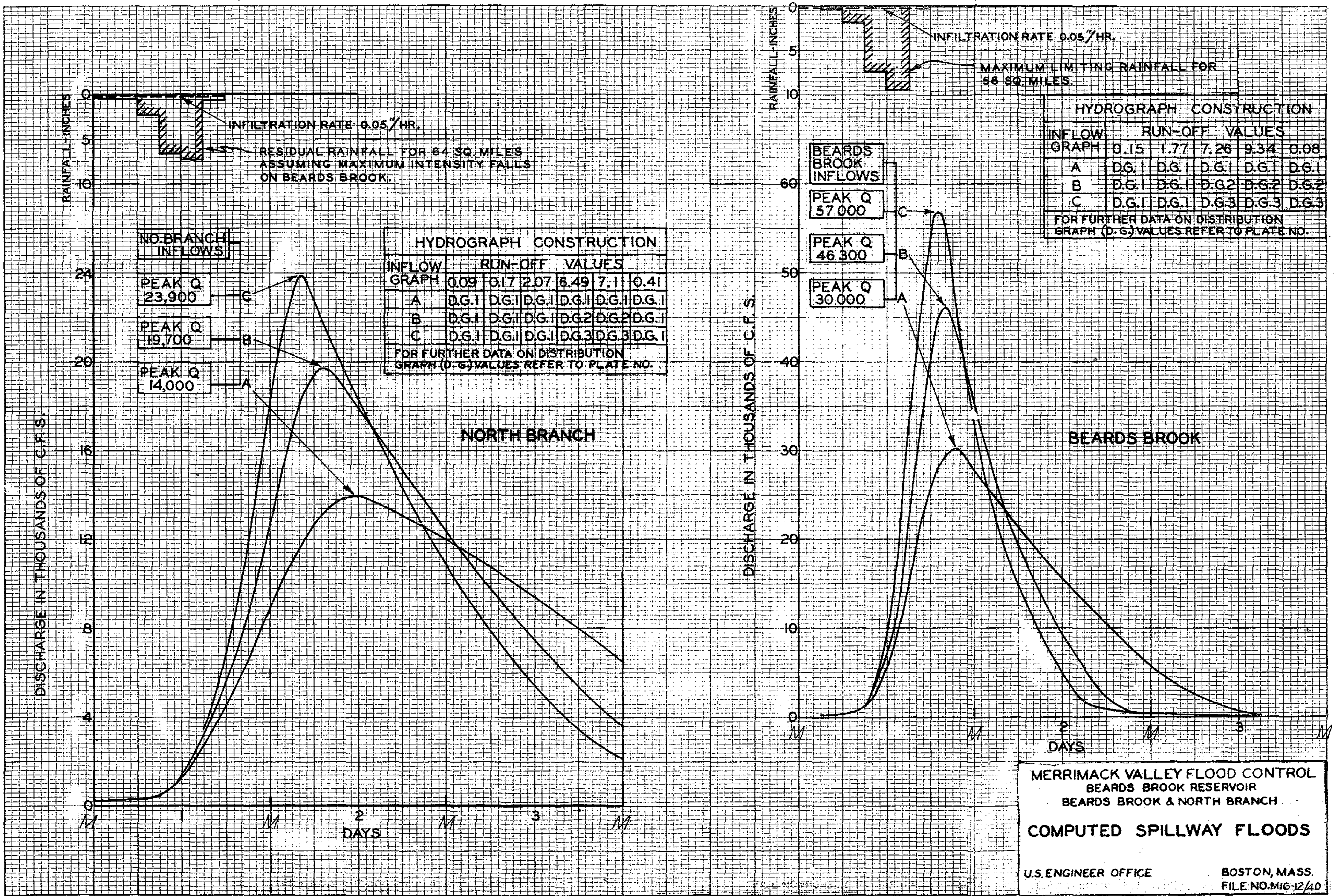


MERRIMACK VALLEY FLOOD CONTROL
CONTOOCOOK RIVER BASIN

CURVES OF LIMITING RAINFALL

U.S. ENGINEER OFFICE | BOSTON, MASS.
FILE NO. 100-50/5 | 18 APRIL 1945

PLATE I-14



HYDROGRAPH CONSTRUCTION (North Branch)

INFLOW GRAPH	RUN-OFF VALUES					
	0.09	0.17	2.07	6.49	7.1	0.41
A	D.G.1	D.G.1	D.G.1	D.G.1	D.G.1	D.G.1
B	D.G.1	D.G.1	D.G.1	D.G.2	D.G.2	D.G.1
C	D.G.1	D.G.1	D.G.1	D.G.3	D.G.3	D.G.1

FOR FURTHER DATA ON DISTRIBUTION GRAPH (D.G.) VALUES REFER TO PLATE NO.

HYDROGRAPH CONSTRUCTION (Beards Brook)

INFLOW GRAPH	RUN-OFF VALUES				
	0.15	1.77	7.26	9.34	0.08
A	D.G.1	D.G.1	D.G.1	D.G.1	D.G.1
B	D.G.1	D.G.1	D.G.2	D.G.2	D.G.2
C	D.G.1	D.G.1	D.G.3	D.G.3	D.G.3

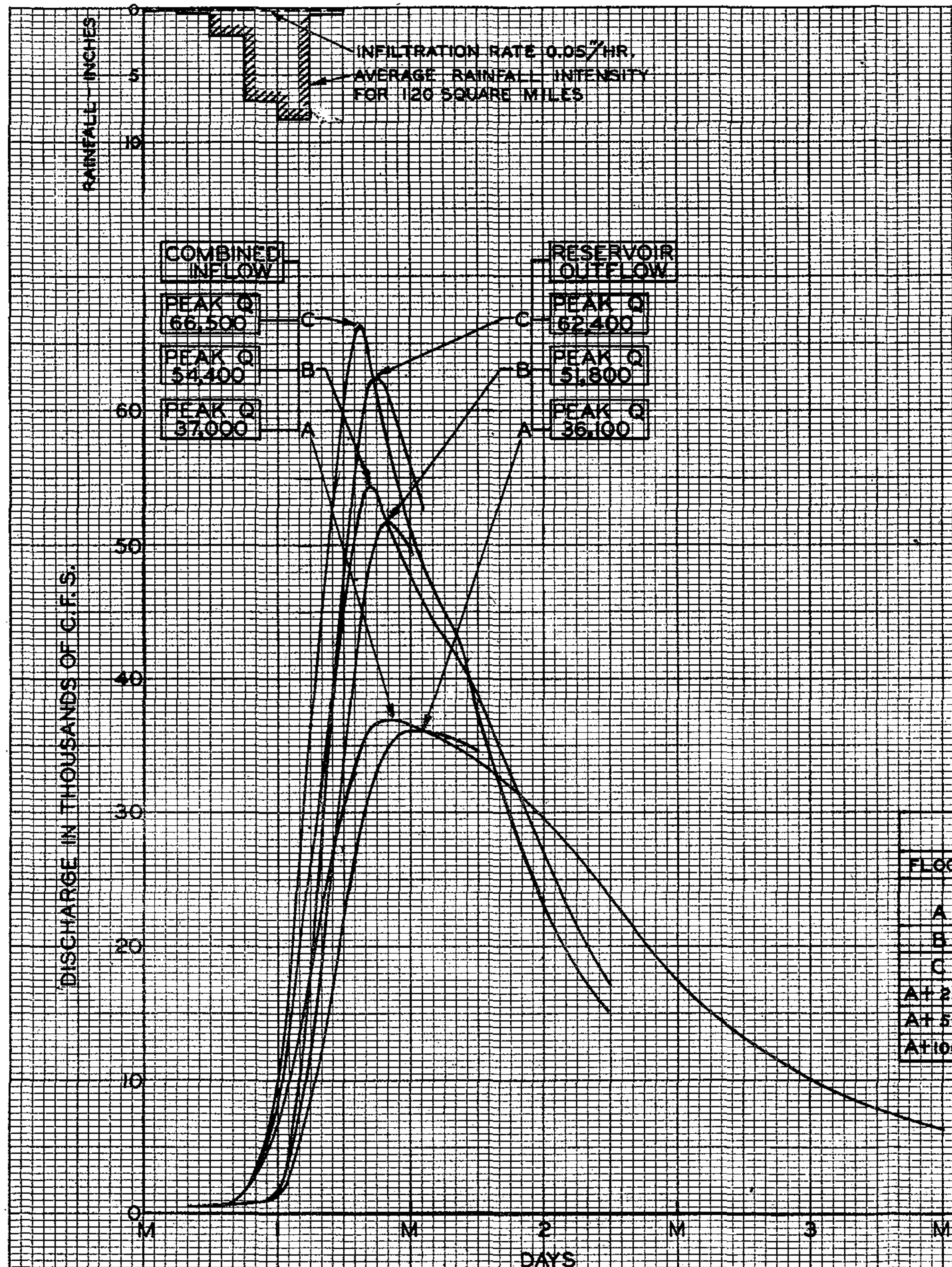
FOR FURTHER DATA ON DISTRIBUTION GRAPH (D.G.) VALUES REFER TO PLATE NO.

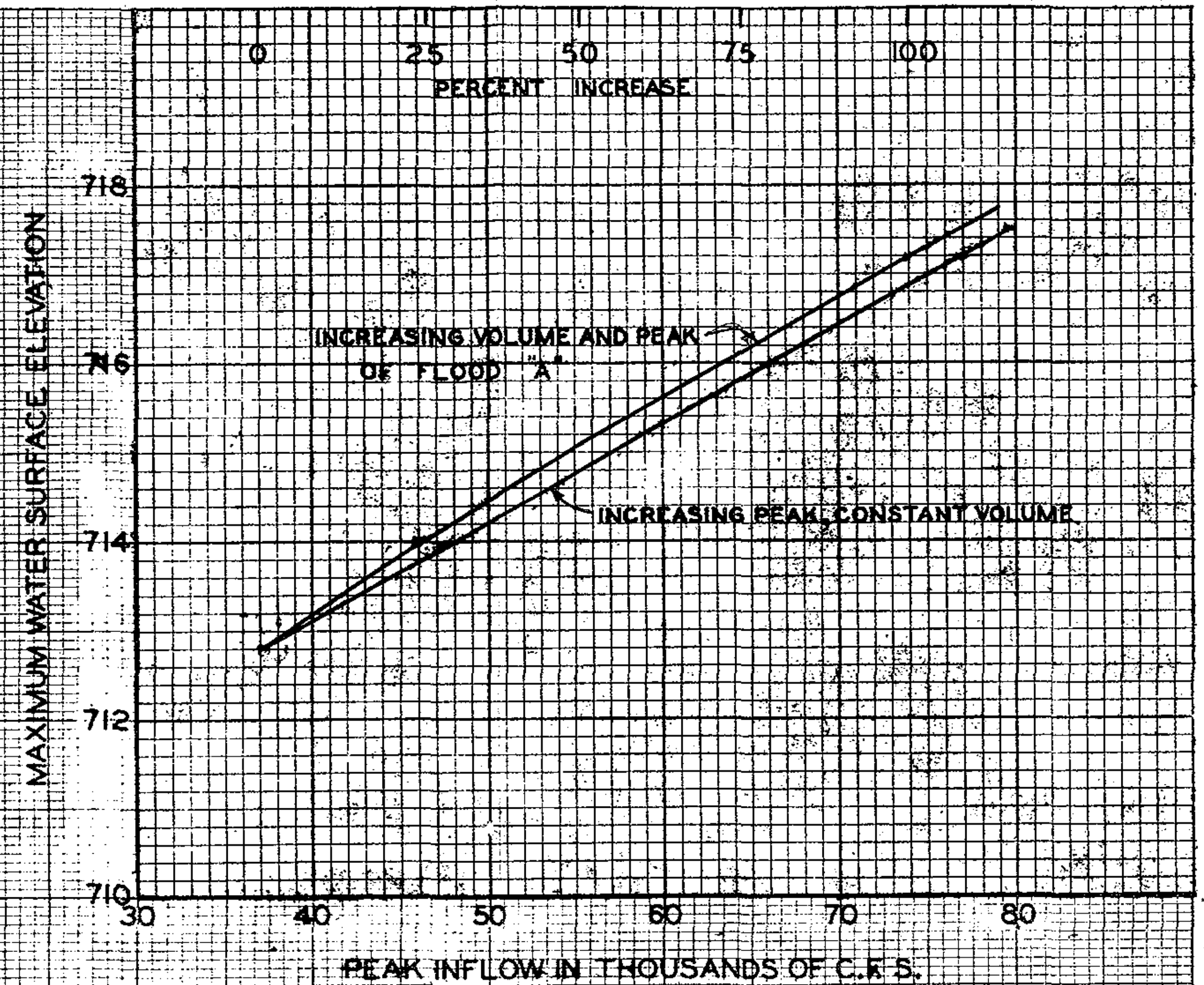
MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

COMPUTED SPILLWAY FLOODS

U.S. ENGINEER OFFICE — BOSTON, MASS.
FILE NO. M16-12/40

PLATE I-15





SPILLWAY FLOODS

FLOOD	VOLUME INCHES	PEAK INFLOW CU. FT. PER. SEC.	PEAK OUTFLOW CU. FT. PER. SEC.	MAX. W.S. ELEV. FEET
A	17.40	37,000	36,100	712.80
B	17.40	54,400	51,800	714.70
C	17.40	66,500	62,400	716.00
A+25%	21.75	46,200	45,500	714.00
A+50%	26.10	55,500	54,600	715.10
A+100%	34.80	74,000	73,000	717.20

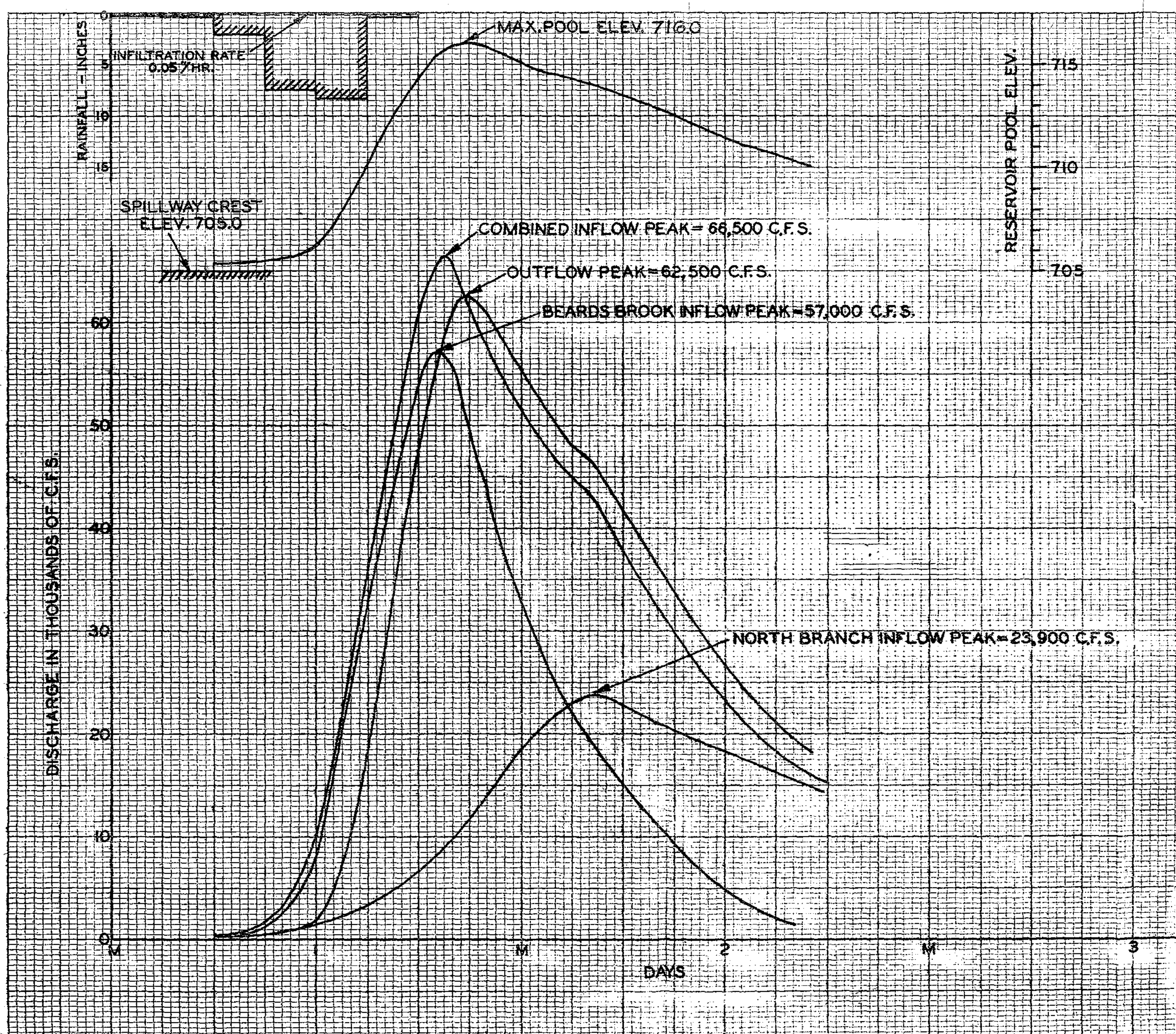
MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

COMBINED COMPUTED SPILLWAY FLOODS

U.S. ENGINEER OFFICE

BOSTON, MASS.
FILE NO. M16-12/41

PLATE I-16



Notes:
The reservoir pool elevation refers to water stage elevation in the North Branch section of the Reservoir. The pool elevation is slightly higher in the Beards Brook section and is analyzed in appendix III.

The rainfall shown graphically is the average intensity for 120 square miles.

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

SELECTED SPILLWAY DESIGN FLOO[D]

U.S. ENGINEER OFFICE
BOSTON, MASS.
FILE NO. M16-12/[...]

PLATE I-

WAR DEPARTMENT

CORPS OF ENGINEERS, U. S. ARMY

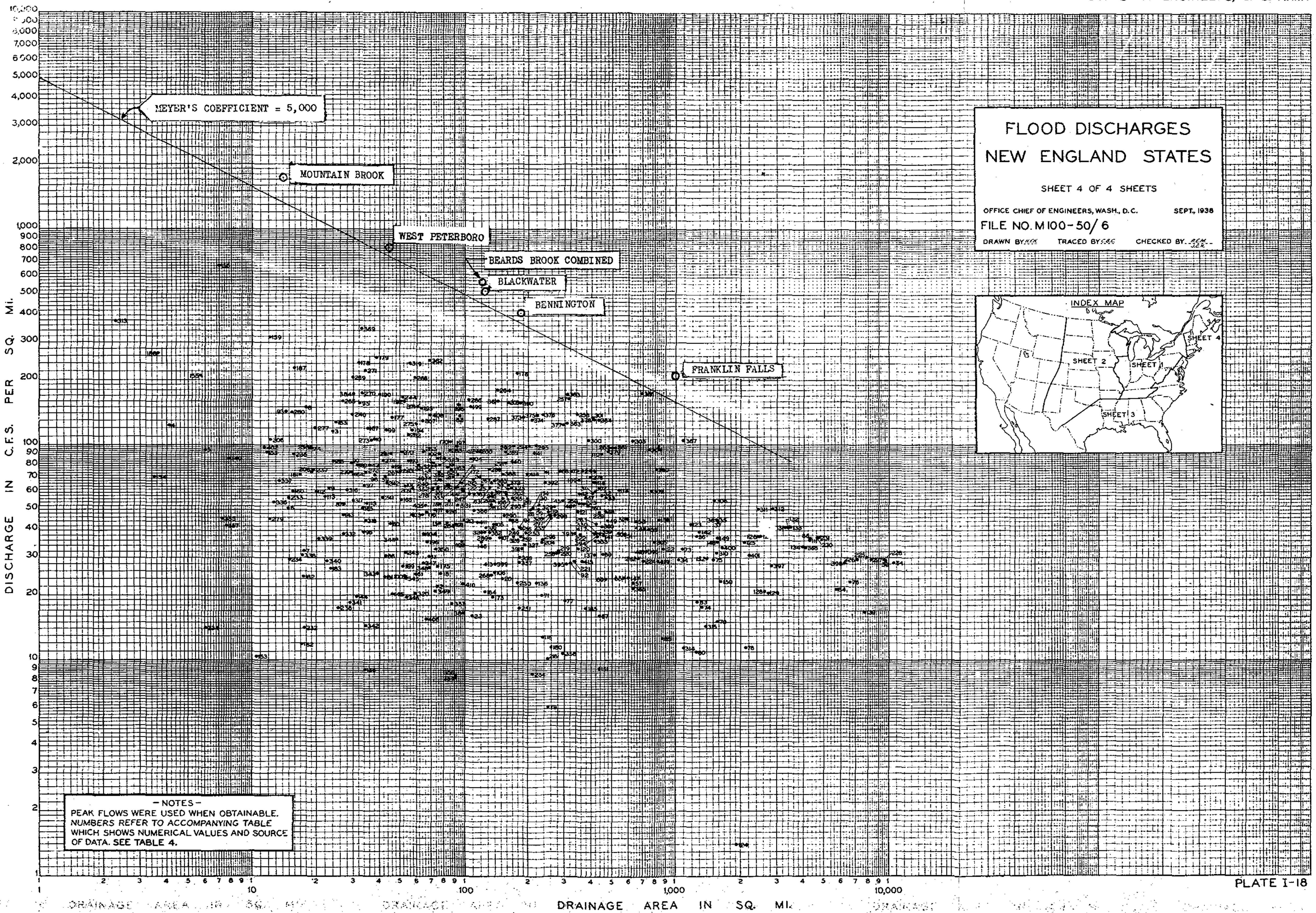


PLATE I-18

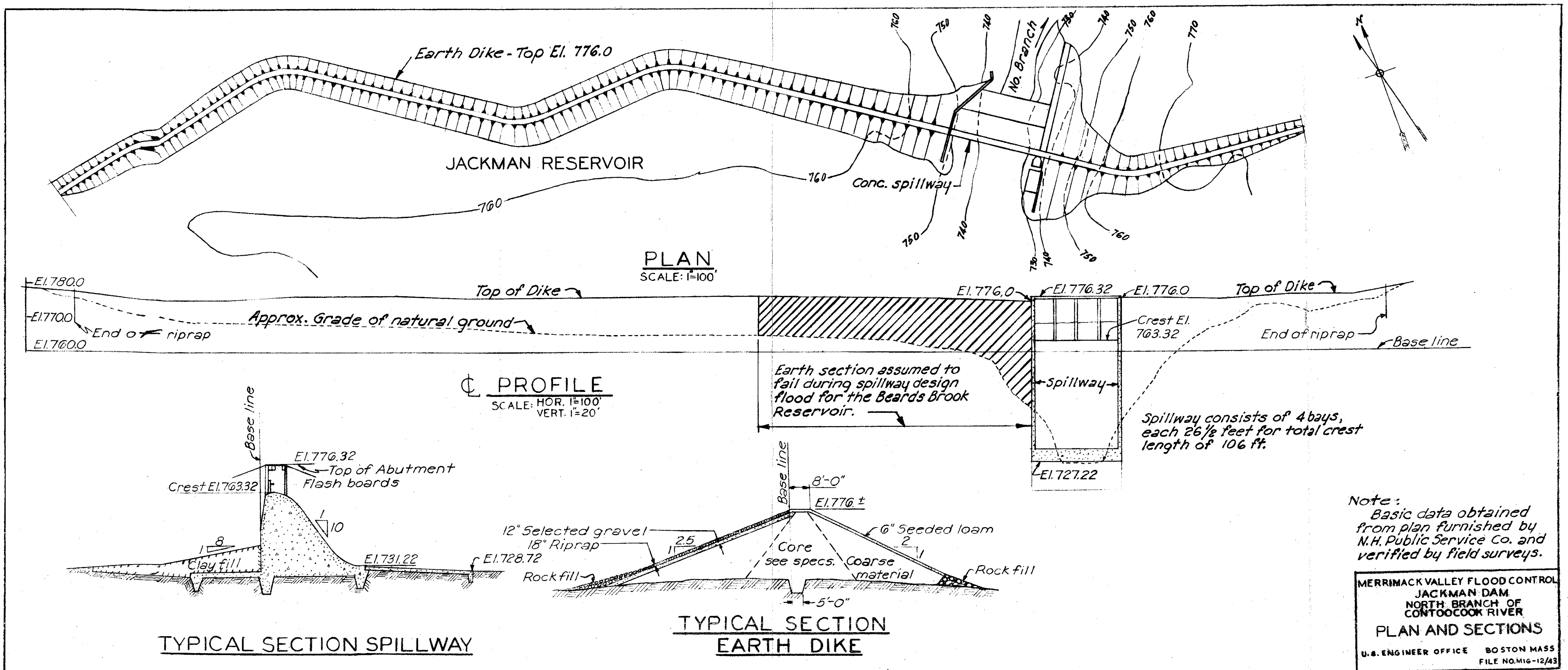
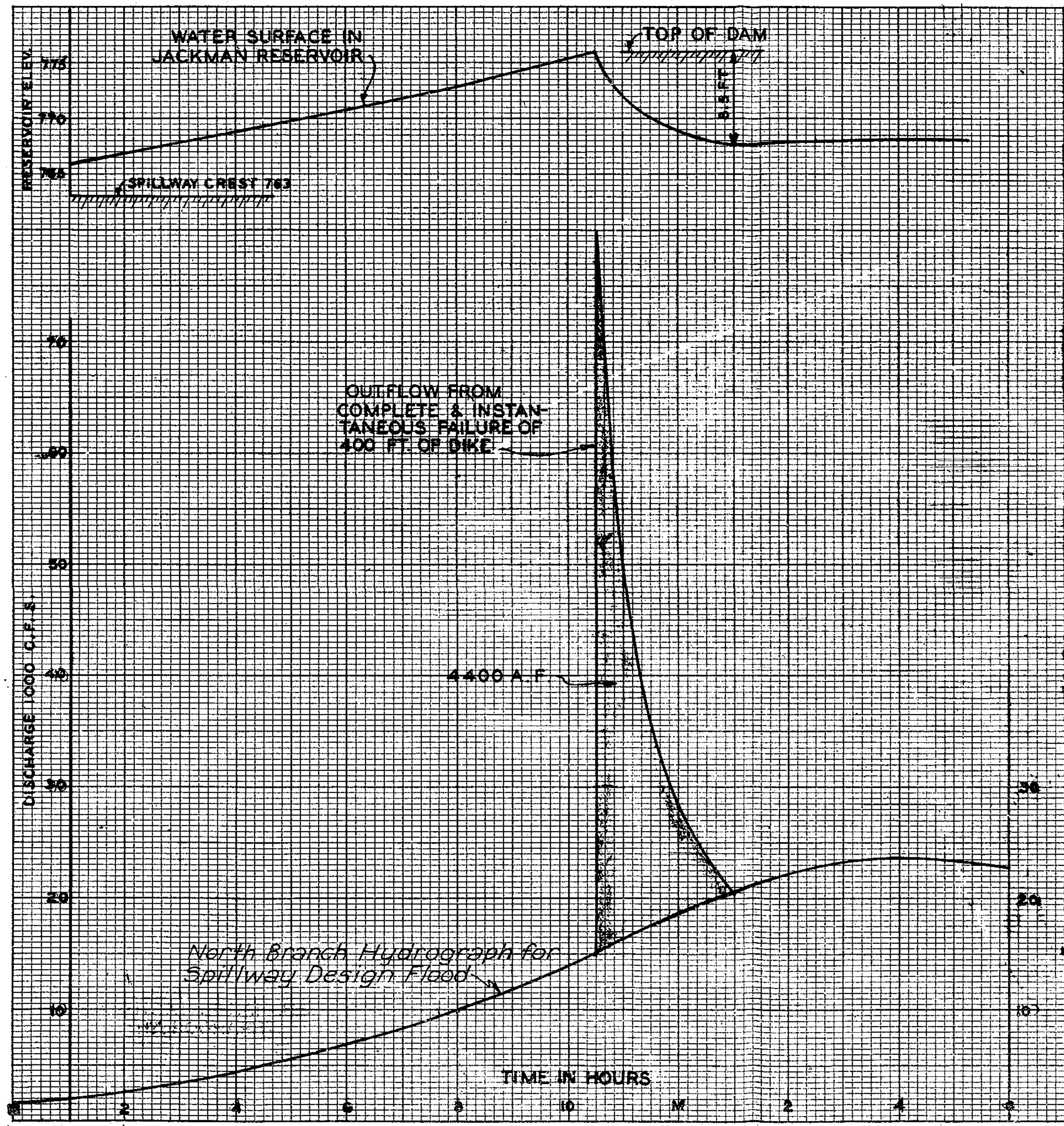
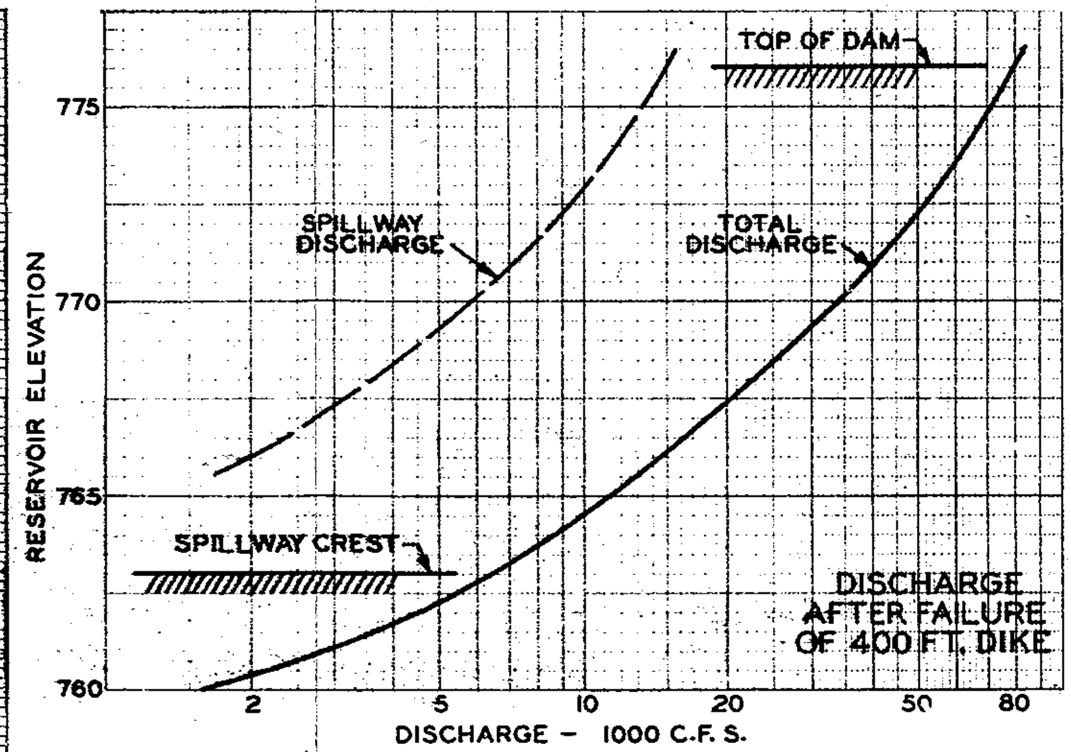
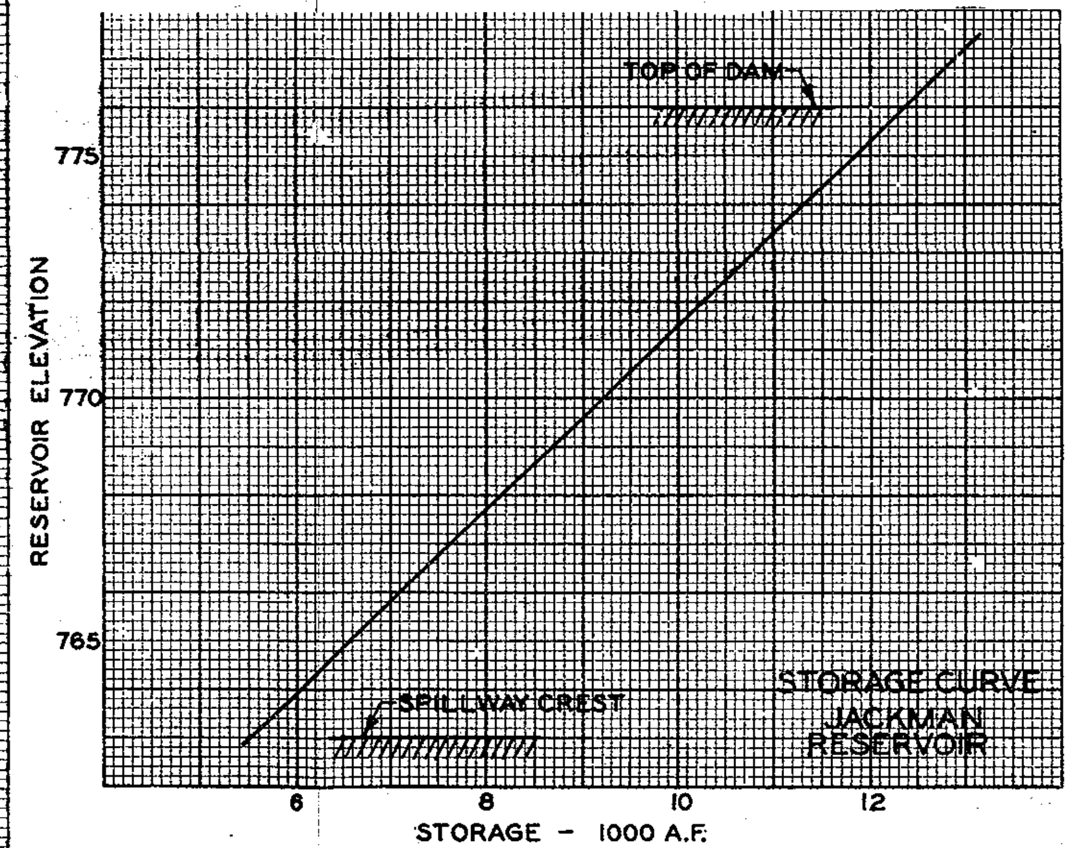

Earth Dike - Top El. 776.0
JACKMAN RESERVOIR
760
750
740
730
770
No. Branch
Conc. spillway
PLAN
SCALE: 1"=100'
El. 780.0
El. 770.0
El. 760.0
End of riprap
Top of Dike
Approx. Grade of natural ground
El. 776.0
El. 776.32
El. 776.0
Crest El. 763.32
End of riprap
Base line
Earth section assumed to fail during spillway design flood for the Beards Brook Reservoir.
Spillway
El. 727.22
Spillway consists of 4 bays, each 26½ feet for total crest length of 106 ft.
℄ PROFILE
SCALE: HOR. 1"=100' VERT. 1"=20'
Base line
El. 776.32
Top of Abutment
Flash boards
Crest El. 763.32
1
10
1
8
Clay fill
El. 731.22
El. 728.72
TYPICAL SECTION SPILLWAY
Base line
8'-0"
El. 776.±
12" Selected gravel
18" Riprap
2.5
1
Core see specs.
Coarse material
6" Seeded loam
2
1
Rock fill
Rock fill
5'-0"
TYPICAL SECTION EARTH DIKE
Note: Basic data obtained from plan furnished by N.H. Public Service Co. and verified by field surveys.
MERRIMACK VALLEY FLOOD CONTROL
JACKMAN DAM
NORTH BRANCH OF CONTOOCOOK RIVER
PLAN AND SECTIONS
U.S. ENGINEER OFFICE BOSTON MASS
FILE NO. M16-12/43

PLATE I-19







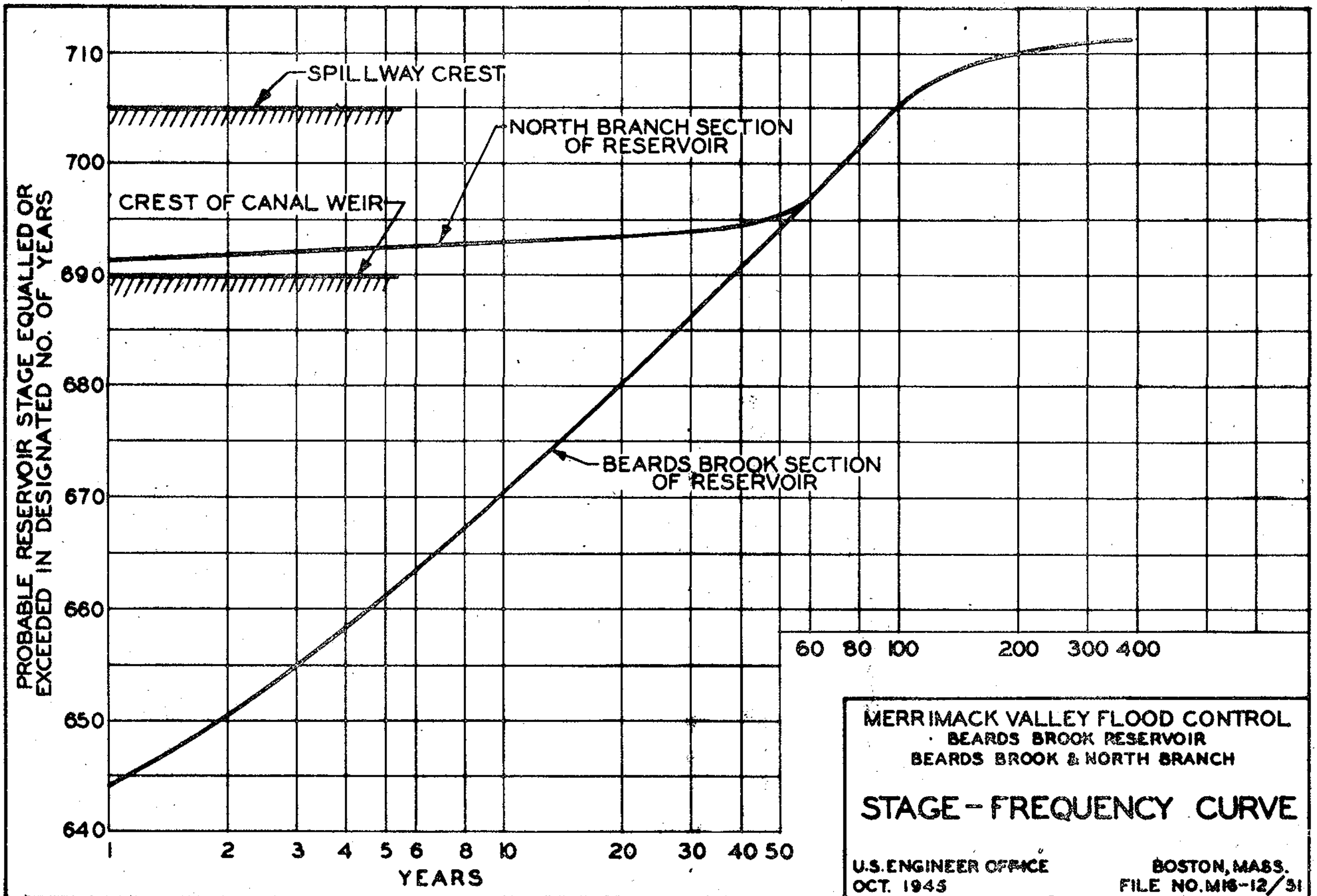
MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

JACKMAN RESERVOIR

HYDRAULIC DATA FOR DAM FAILURE

U.S. ENGINEER OFFICE — BOSTON, MASS.
OCT. 1945 — FILE NO. M16-12/44

PLATE I - 20

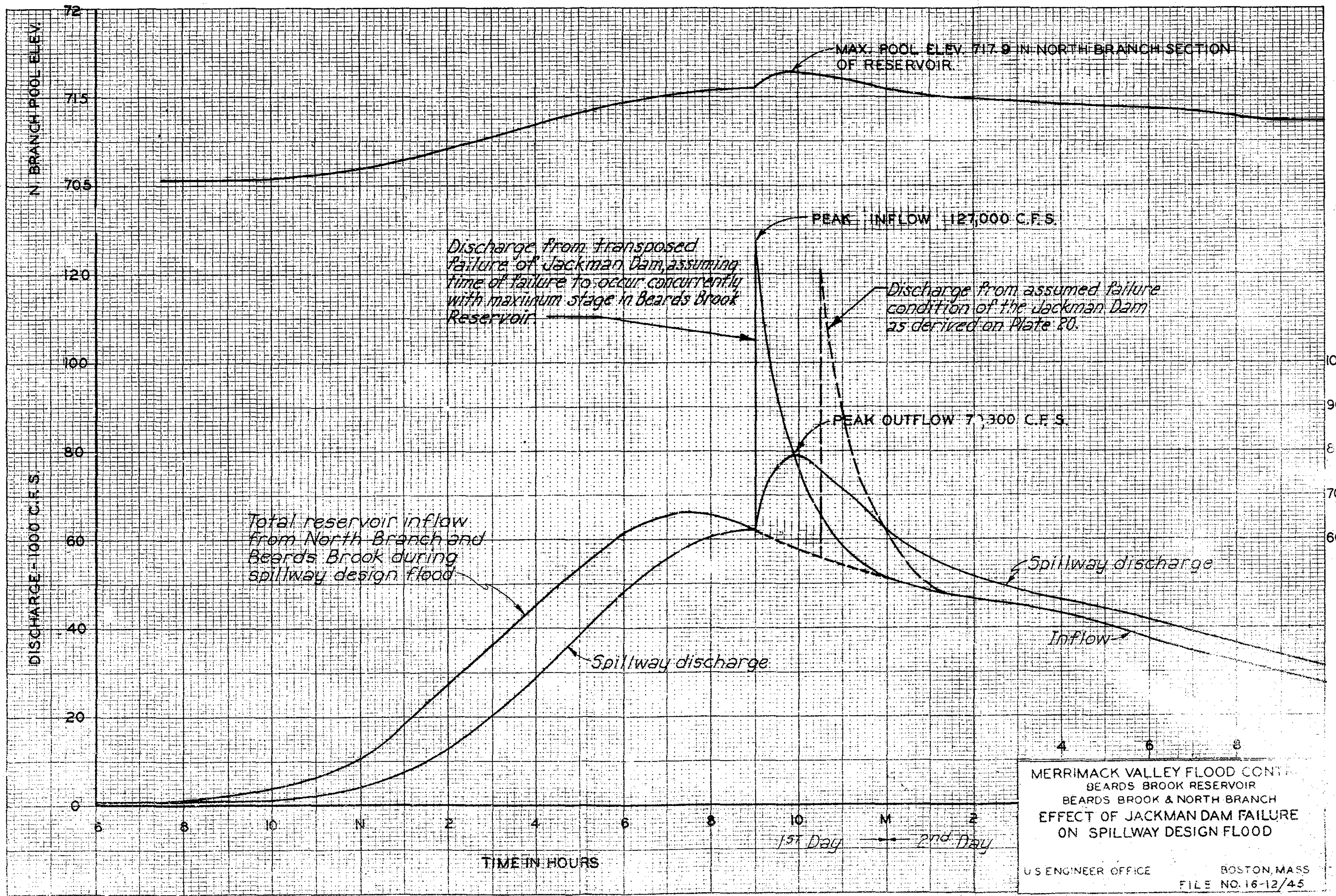


MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

STAGE - FREQUENCY CURVE

U.S. ENGINEER OFFICE
OCT. 1945

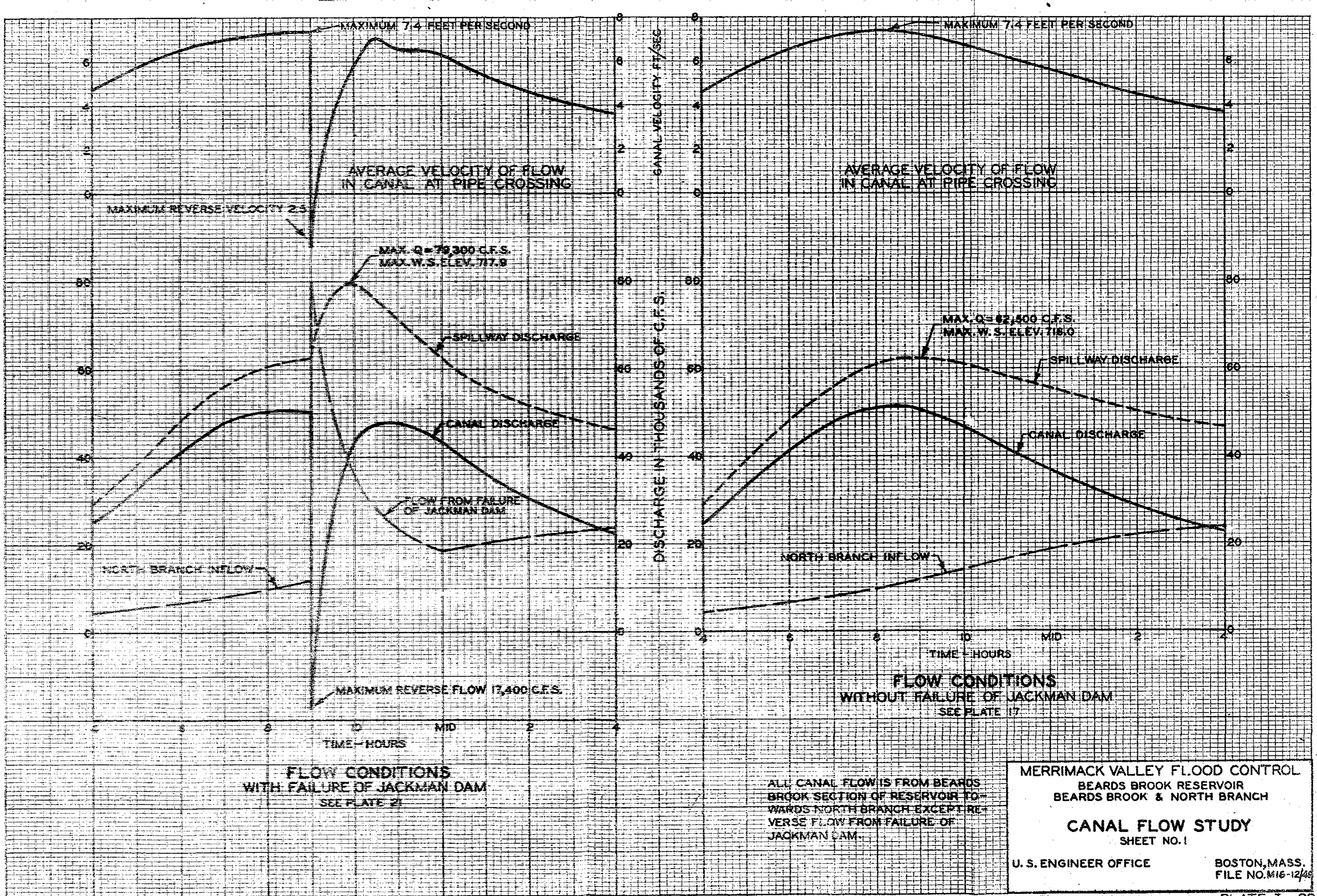
BOSTON, MASS.
FILE NO. M16-12/31

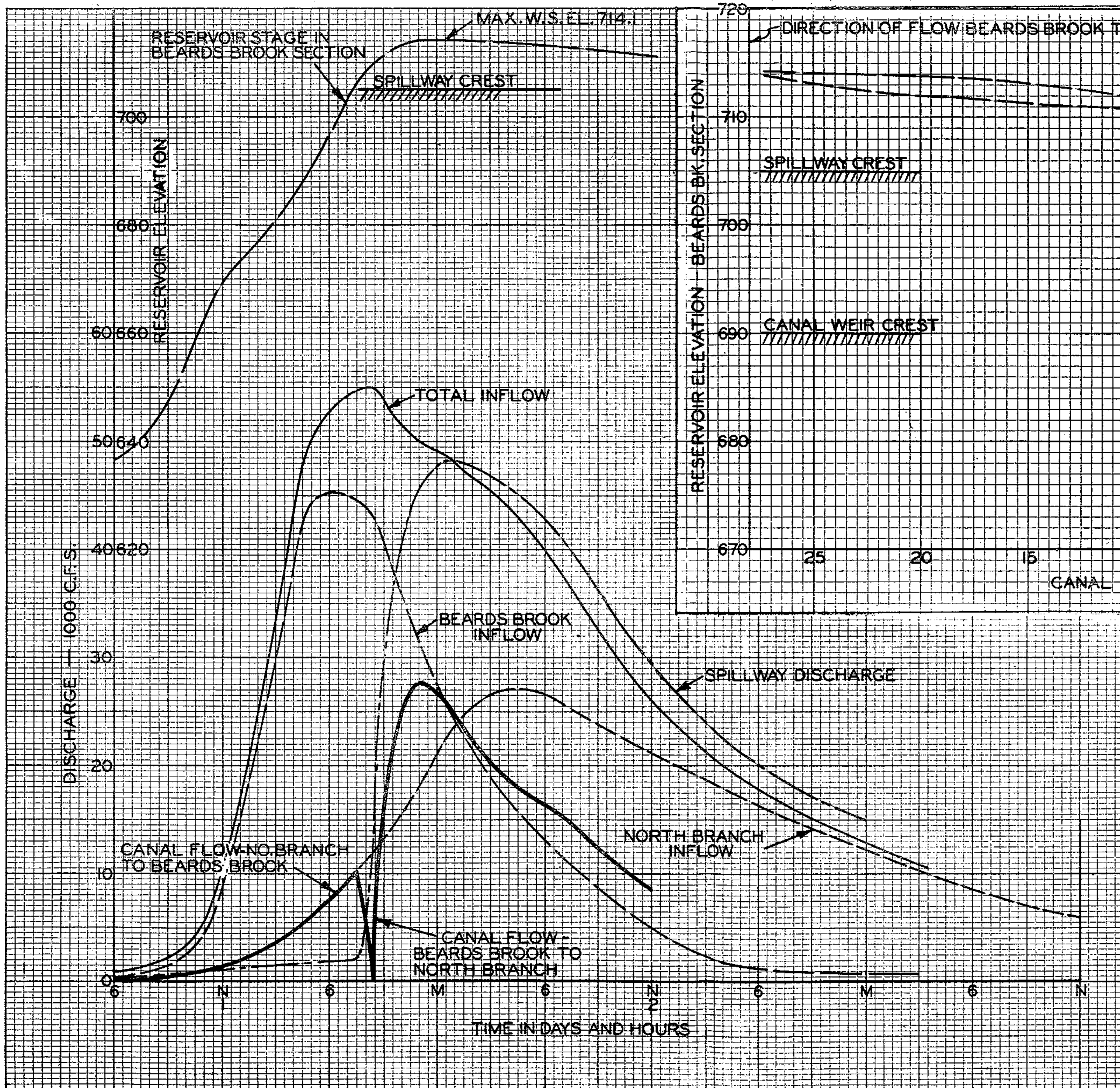


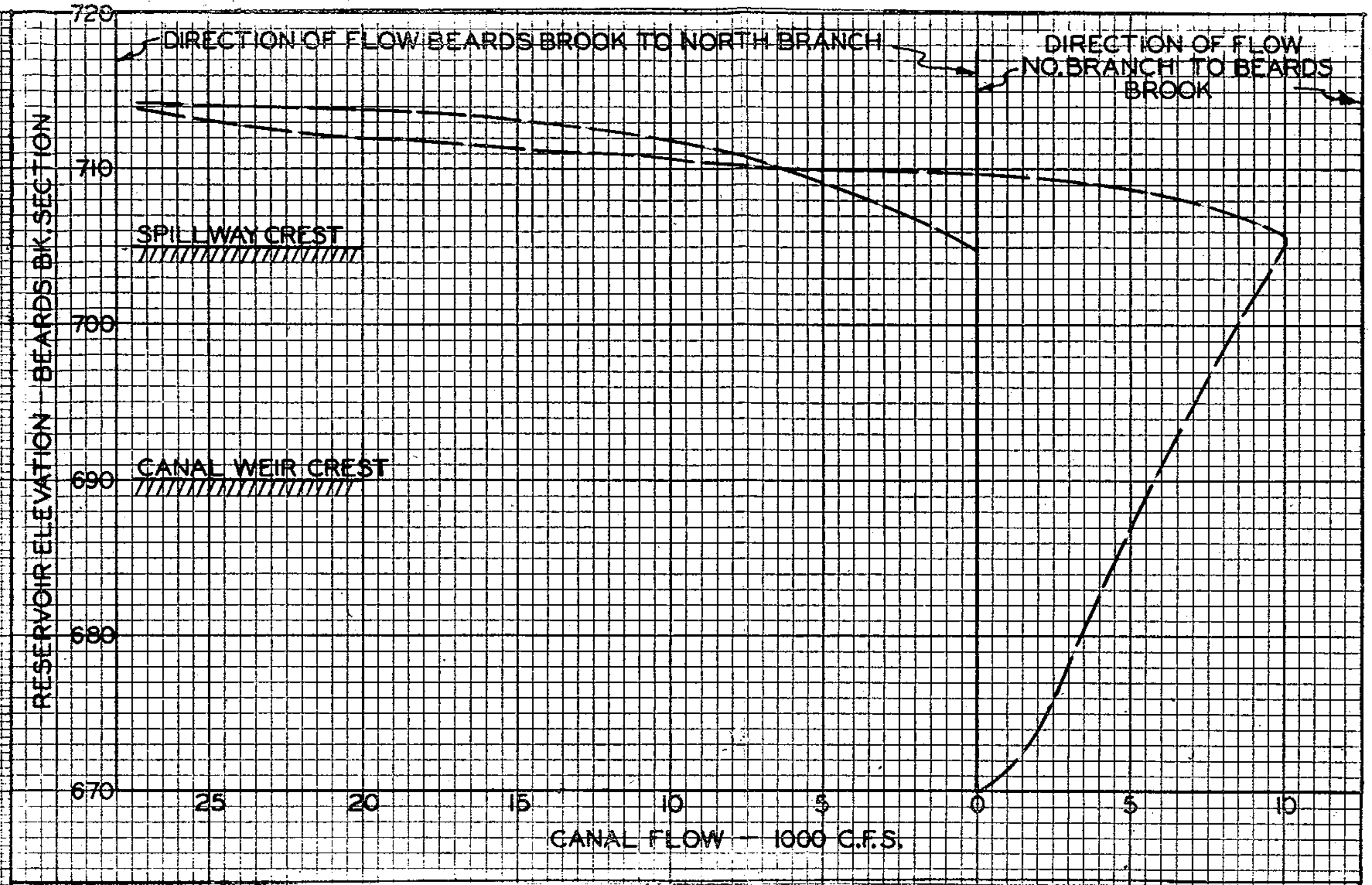
MERRIMACK VALLEY FLOOD CONT
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH
EFFECT OF JACKMAN DAM FAILURE
ON SPILLWAY DESIGN FLOOD

U.S. ENGINEER OFFICE BOSTON, MASS.
FILE NO. 16-12/45

PLATE I-2


MAXIMUM 7.4 FEET PER SECOND
AVERAGE VELOCITY OF FLOW IN CANAL AT PIPE CROSSING
MAXIMUM REVERSE VELOCITY 2.5
CANAL VELOCITY FT/SEC
MAX. Q = 79,300 C.F.S.
MAX. W.S. ELEV. 717.9
SPILLWAY DISCHARGE
CANAL DISCHARGE
FLOW FROM FAILURE OF JACKMAN DAM
NORTH BRANCH INFLOW
MAXIMUM REVERSE FLOW 17,400 C.F.S.
DISCHARGE IN THOUSANDS OF C.F.S.
TIME - HOURS
FLOW CONDITIONS
WITH FAILURE OF JACKMAN DAM
SEE PLATE 21
MAXIMUM 7.4 FEET PER SECOND
AVERAGE VELOCITY OF FLOW IN CANAL AT PIPE CROSSING
MAX. Q = 62,500 C.F.S.
MAX. W. S. ELEV. 718.0
SPILLWAY DISCHARGE
CANAL DISCHARGE
NORTH BRANCH INFLOW
TIME - HOURS
FLOW CONDITIONS
WITHOUT FAILURE OF JACKMAN DAM
SEE PLATE 17
ALL CANAL FLOW IS FROM BEARDS BROOK SECTION OF RESERVOIR TOWARDS NORTH BRANCH EXCEPT REVERSE FLOW FROM FAILURE OF JACKMAN DAM.
MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH
CANAL FLOW STUDY
SHEET NO. 1
U. S. ENGINEER OFFICE
BOSTON, MASS.
FILE NO. M16-12/48





Reservoir and Storm Assumptions

1. *Reservoir empty at beginning of storm.*
2. *Ungated outlet in spillway inoperative.*
3. *Beards Brook conduit discharging with 7'x 4' gate open until start of spillway discharge. All gates are then closed.*
4. *Flood caused by storm based on limiting rainfall curves (Pl. 14) with assumption that maximum intensity falls on the North Branch drainage area. The unit hydrographs used for the spillway design flood are applicable. The rainfall intensities are as follows:*

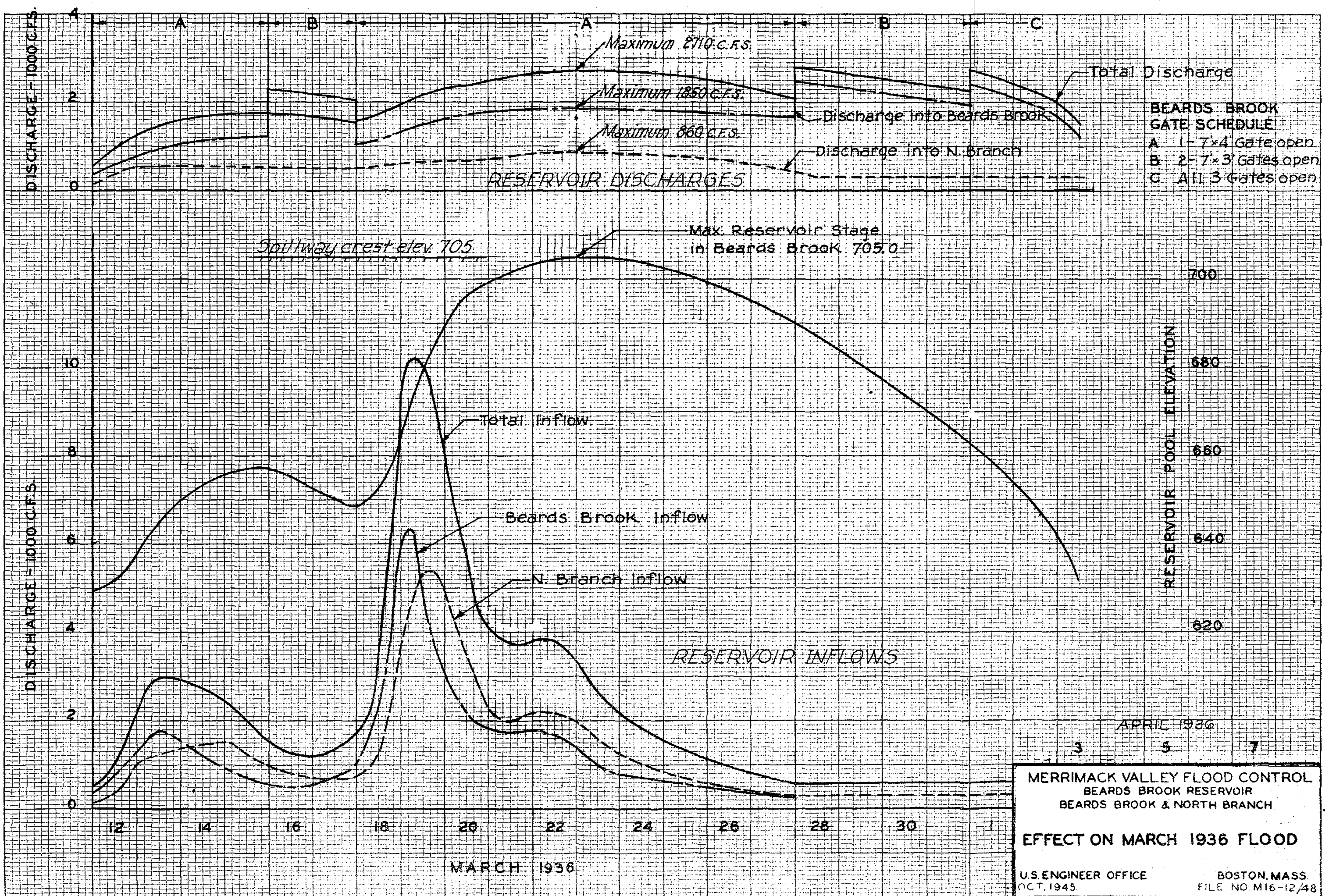
Hours	Beards Brook	North Branch
3	0.29 Inches	0.12 Inches
6	0.31 "	0.85 "
9	2.26 "	1.92 "
12	6.56 "	7.38 "
15	7.16 "	9.30 "
18	0.54 "	0.35 "
24	0.08 "	0.08 "
Total	17.20	19.40

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

CANAL FLOW STUDY
SHEET NO. 2

U.S. ENGINEER OFFICE — BOSTON, MASS.
FILE NO. 16-12/47

PLATE I-23

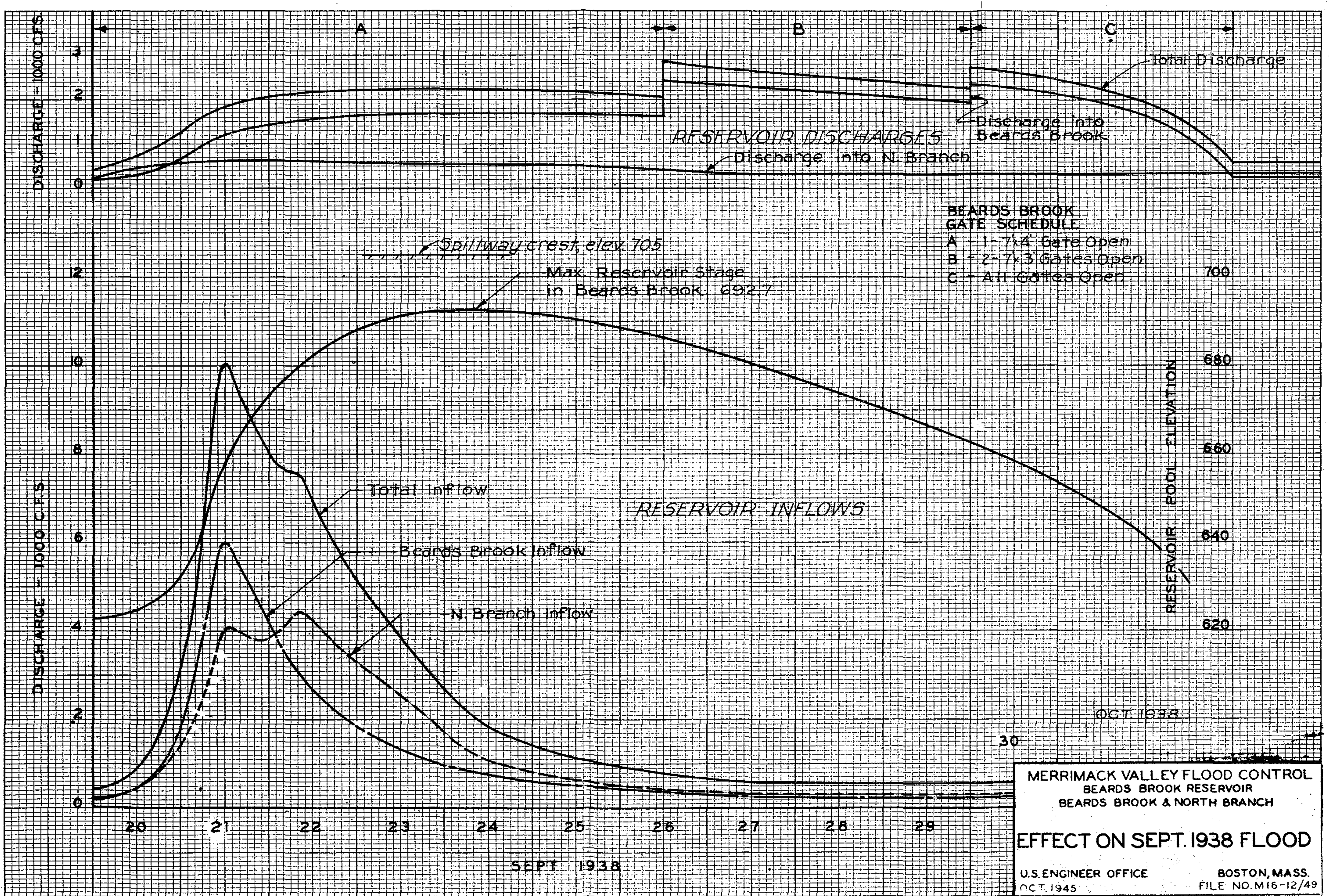


MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

EFFECT ON MARCH 1936 FLOOD

U.S. ENGINEER OFFICE — BOSTON, MASS.
OCT. 1945 — FILE NO. M16-12/48

PLATE I - 24



MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

EFFECT ON SEPT. 1938 FLOOD

U.S. ENGINEER OFFICE BOSTON, MASS.
OCT. 1945 FILE NO. M16-12/49

PLATE I-25

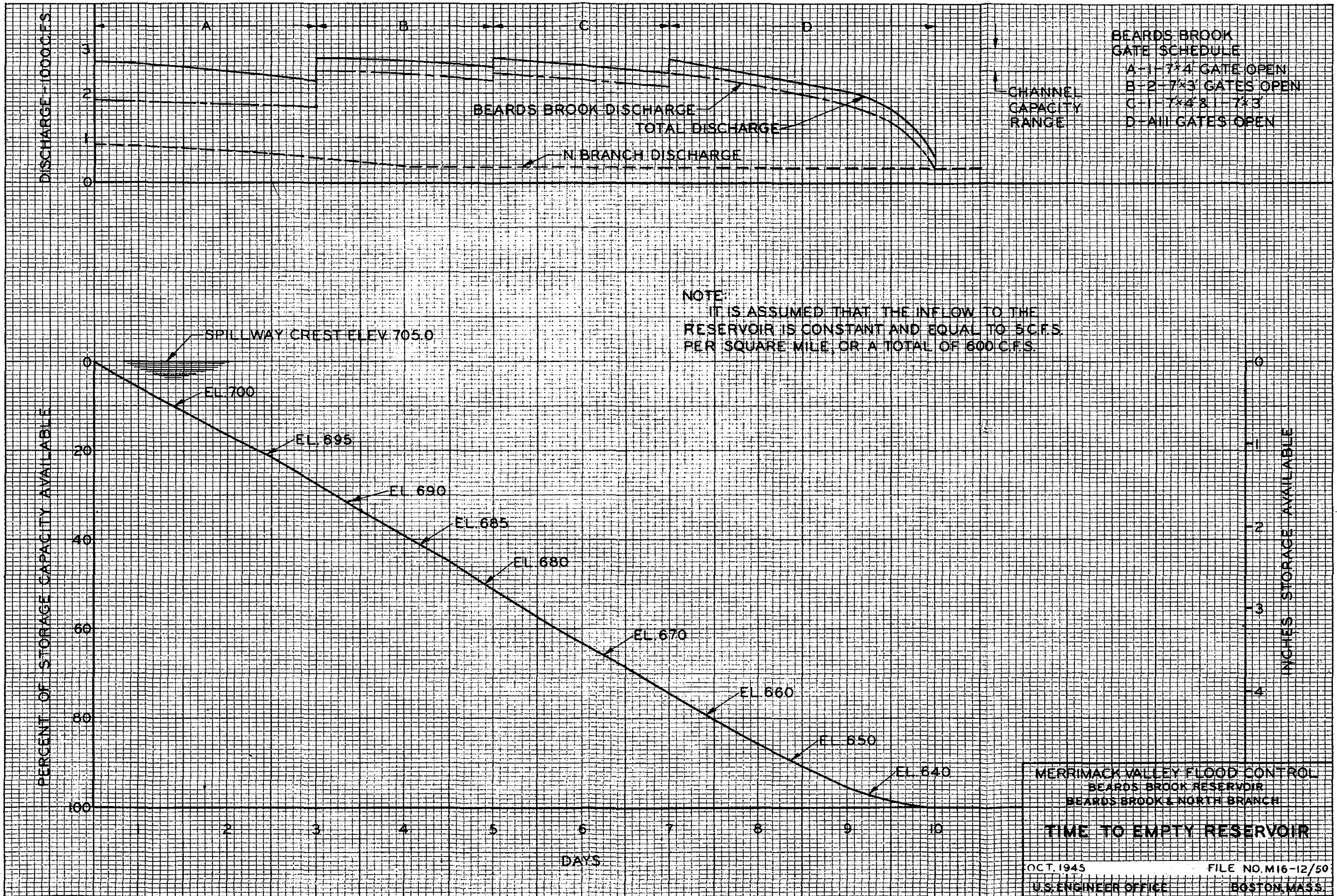

DISCHARGE-1000 C.F.S.
A
B
C
D
BEARDS BROOK
GATE SCHEDULE
A-1-7'x4' GATE OPEN
B-2-7'x3' GATES OPEN
C-1-7'x4' & 1-7'x3'
D-All GATES OPEN
CHANNEL
CAPACITY
RANGE
BEARDS BROOK DISCHARGE
TOTAL DISCHARGE
N. BRANCH DISCHARGE
NOTE:
IT IS ASSUMED THAT THE INFLOW TO THE RESERVOIR IS CONSTANT AND EQUAL TO 5 C.F.S. PER SQUARE MILE, OR A TOTAL OF 600 C.F.S.
SPILLWAY CREST ELEV. 705.0
EL. 700
EL. 695
EL. 690
EL. 685
EL. 680
EL. 670
EL. 660
EL. 650
EL. 640
PERCENT OF STORAGE CAPACITY AVAILABLE
INCHES STORAGE AVAILABLE
DAYS
MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH
TIME TO EMPTY RESERVOIR
OCT. 1945
FILE NO. M16-12/50
U.S. ENGINEER OFFICE
BOSTON, MASS.

PLATE I-26

APPENDIX II

GEOLOGY, SOIL DATA AND ANALYSES

To accompany definite project report
dated November 1945

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX II - GEOLOGY, SOIL DATA AND ANALYSES

CONTENTS

PLATES

DEFINITE PROJECT REPORT
BEARDS BROOK RESERVOIR

APPENDIX II

GEOLOGY, SOIL DATA AND ANALYSES

a. Introduction.- This appendix presents the results of investigations covering field explorations, laboratory testing and engineering analyses of soils and rock for foundation and embankment purposes in connection with the Beards Brook flood control project. Investigations and analyses have been performed in accordance with the requirements of the Civil Works Manual of the Office, Chief of Engineers, using procedures of accepted standard practice. The results of the investigations indicate that the foundation conditions are satisfactory for the construction of an earth dam with spillway in bedrock on one valley wall; that adequate suitable materials for embankment construction are readily available; that the foundation and embankment possess ample stability, and that seepage through and under the embankment will be negligible.

b. Exploration.- (1) Reconnaissance.- Reconnaissance was conducted in the general vicinity of the site selected for the project location. Topography and geography were studied and surficial examination was made to investigate the principal geological conditions. Observations indicated that a satisfactory foundation existed and that suitable construction materials were available within reasonable hauling distance.

(2) Seismic Exploration.- Seismic exploration was conducted under the direction of Mr. E. R. Shepard, Office, Chief of Engineers, at one location on the foundation of the dam (see Plate II-3) and at four locations in the area proposed for the impervious borrow areas (see Plate II-7). Bedrock was located in the borrow area location but not located at the line fired on the foundation of the dam. Some indication of the character of the overburden materials was obtained by wave velocities above the rock surface.

(3) Methods of Subsurface Exploration.- (a) Drill Holes.- Explorations to recover soil and rock samples for classification, and determination of extent of occurrence, were performed in 3-inch drill holes from which 2-inch soil samples were recovered by drive sampling into undisturbed

material, and from which 1-5/8" rock cores were recovered using a diamond drill. Smaller soil samples were obtained from 2-inch holes for borrow exploration.

(b) Test Pits.- Shallow test pits approximately six feet deep were used to obtain large samples for determination of natural soil properties and for use in laboratory determination of remolded properties. Test pits were extended by auger boring where possible. Where large samples were required from depths of more than six feet below ground surface, test pits were sheathed and braced as required.

(c) Pressure Tests in Rock.- Where cores from bedrock indicated weathered and fractured rock, and where additional information was desired about possible underseepage in the rock, drill holes were pressure tested for total leakage and then for leakage along five foot segments. Pressures used in testing the rock were approximately one pound per square inch for every foot in depth below ground surface.

(d) Observation Wells.- Observation wells have been installed in drill holes at points where a knowledge of ground water conditions is desired.

(4) Foundation Investigation at Dam Site.- (a) Foundation exploration at the dam site consisted of thirty-eight 3-inch drill holes, five shallow test pits and two deep test pits. Location of explorations is shown on Plate II-2 and II-3, and logs of individual explorations are shown on Plates II-4 and II-5. Samples were obtained from all explorations for classification and undisturbed samples were obtained from test pits for determination of natural and remolded soil properties.

(b) Pressure tests in bedrock were performed in four holes at this site. In each hole tested approximately the upper five feet of rock was moderately fractured and the pressure testing equipment could not be sealed satisfactorily. Below five feet, the rock is fresh and moderately fractured, with no serious leakage encountered in any of the holes tested.

(c) Observation wells have been installed in Drill Holes 33 and 38 where artesian flow was encountered during drilling operations. The source of the flow is a large body of sand approximately 70 feet below ground surfaco, and which extends to bedrock at a depth of more than 110 feet. Hydrostatic head at the Drill Hole 33 was over 17 feet above the ground surface and a flow of 12 gallons per minute was measured at the ground surface at date of exploration. After 6 momths of continuous discharge, the flow had been reduced to approximately one gallon per minute. This reduction in flow is believed to have been caused by sand which has entered the bottom of the well and impeded the flow of water. Exploration at D38 was completed in July 1945 and similar artesian conditions were encountered during drilling.

(5) Borrow Exploration.- Exploration for suitable borrow materials consisted of a field reconnaissance, supplemented by the drilling of sixteen 2-inch drill holes and the digging of four shallow test pits. The drill holes and test pits explored to date were located within proposed pervious borrow areas. Seismic exploration in the proposed impervious borrow area has been conducted and exploratory drilling has been scheduled. Samples of each soil stratum encountered in drill holes and test pits were obtained for classification, and representative samples of principal materials encountered in excavation of test pits were obtained for laboratory determination of soil properties. Natural density of pervious materials was determined by sand displacement method in the field. A plan of borrow exploration is shown on Plate II-7 and a record of individual explorations is shown on Plate II-8.

c. Geology.- (1) Regional.- (a) Beards Brook Valley.- The valley of Beards Brook is a tributary of the Contoocook River valley and shares its geologic background. The rocks of the Beards Brook Valley are metamorphic and igneous of Paleozoic Age. The metamorphic rocks are schists and gneisses which were deposited originally as flat or gently sloping beds beneath the water surface. A period of diastrophism which occurred subsequent to this deposition resulted in the crumbling and folding of the layers until they appeared at every attitude. Large amounts of magna were injected into the metamorphosed rocks, where relatively slow cooling occurred and granite or related granitic rocks formed. A period of regional uplift followed and streams began the gradual erosion of the overlying metamorphic rocks. As the drainage pattern developed the streams cut downward through the metamorphic rocks and exposed large areas of the granite. At the beginning of the Pleistocene Period, the Beards Brook Valley had been developed and the brook was flowing in a bedrock channel. During the

Pleistocene Period, the valley was covered by a portion of the continental glacier that occupied Canada and the northern part of the United States. The glacier modified existing topography by erosion and by deposition. Erosion by the glacier consisted of bevelling the northern slopes of hills and steepening their southern slopes. In addition, it scoured and widened north-south valleys and tended to deepen and widen saddles transverse to its line of motion. Deposition by the glacier caused the greatest change in topography. Glacial deposits are of two types: the unsorted, unstratified glacial till deposited directly from the ice; and the sorted gravels, sands, silts and clays that are washed from the ice and deposited in water. Glacial till may consist of a heterogeneous mixture of clay, silt, sand, gravel, cobbles and boulders, or some of these sizes may be lacking. In general, the till of the Beards Brook Valley does not have material as fine as the clay sizes in any appreciable amount. The till consists principally of two classes: sandy till which has a predominance of sand sizes and silty till which has a predominance of silt sizes. Glacial till is highly stable, capable of furnishing good support and is practically impervious unless modified by weathering. The till was deposited in elliptical shaped hills known as drumlins, and as blankets which were smeared across the valleys, partially filling them and disorganizing the drainage system. In many cases interstream divides were buried by glacial deposits and streams now pass from one old drainage basin to another. Where streams still flow within the old valley they are seldom in the old channel, therefore, when a stream has a rock floor, there may be a buried channel nearby. The sorted materials consist of gravels, sands, silts and clays or, as in the case of a modified glacial drift, combinations of these sizes. These materials have been washed from the ice, sorted or partially sorted and deposited in the form of terraces, eskers and outwash plains. The Beards Brook Valley has been largely filled with both sorted and unsorted glacial deposits and it is within these deposits that the present channel has been developed.

(b) Vicinity of Dam Site.- In the vicinity of the dam site, Beards Brook flows southward in a channel composed largely of compact glacial till. The present channel is located almost directly over the pre-glacial channel and approximately 100 feet above it. North Branch Brook which is located approximately one half mile west of Beards Brook has a bedrock floor which is located high on the old pre-glacial valley wall. Large quantities of glacio-fluvial material have been deposited within one mile upstream of the dam site; these consist mostly of kame terrace deposits and are composed of coarse to fine sand.

(c) Source of Data.- Information pertaining to historical and existing geology of the Beards Brook Valley has been abstracted from reports on the Beards Brook site presented to the Boston District Office by Mr. Sidney Paige, Geologist, U. S. Engineer Office, North Atlantic Division, New York City; by Mr. Charles P. Berky, consulting geologist, Columbia University; by Mr. Irving B. Crosby, consulting geologist, Boston, Massachusetts.

(2) Dam Site.- (a) General.- The geology of the dam site is discussed in the following sub-paragraphs according to principal conditions encountered. A plan of foundation exploration showing the location of all drill holes, test pits and observation wells is shown on Plates II-2 and II-3 and graphic logs of individual explorations are shown on Plates II-4 and II-5. Geological profiles are shown on Plate II-6.

(b) East Abutment.- The east abutment of the dam is composed of a compact sandy to silty glacial till that overlies bedrock at depths of 50 feet or more. The upper zone of the till to a depth of four to six feet has been affected by weathering and frost action, and is less compact that the underlying till. Boulders of varying sizes occur in abundance on the ground surface. Bedrock rises toward the east but does not outcrop within the immediate vicinity of the dam site.

(c) Valley Bottom.- The valley bottom includes the channel now occupied by Beards Brook where the maximum embankment section occurs. The principal material is a compact sandy to silty till that overlies a body of pervious sand at depth of more than 60 feet. The sand body is extensive and is probably a deposit of pre-glacial sediments that occupy the old channel. Two observation wells were installed, one with bottom of pipe within the sand stratum and one with bottom of pipe in porous rock under the sand. Continuous flow from the sand stratum into these two observation wells indicates that water has ready access to the material. Surface boulders are strewn liberally throughout the area.

(d) West Bank.- With the exception of a short section included under the valley section, the embankment west of Beards Brook is relatively low. The foundation material is compact sandy to silty till with a capping of four to six feet in depth that has been affected by weathering and frost action. Bedrock surface slopes steeply from outcrops in the spillway area to depth of more than 50 feet in the valley bottom. Boulders varying in size from one to fifty cubic yards are abundant in surface concentrations.

(e) Spillway.- The spillway site forms the west abutment of the dam. The site has a thin covering of gravelly, silty sand overlying bedrock at depth of approximately 10 feet. North Branch Brook flows in a general southeasterly direction through the site and has exposed the rock over a large area. The principal rock is moderately fractured porphyritic granite with many inclusions of silicious schist. The rock is fresh and structurally strong throughout the area except for a shallow weathered capping. Adjacent to the downstream side of the spillway site a local zone of weathering occurs to a depth of approximately 40 feet.

(f) Diversion Channel.- The diversion channel is located upstream of the embankment on the west bank of Beards Brook in such a location that it will direct flood flow of North Branch Brook into the principal reservoir area. The material at this location consists of semi-compact, to very compact gravelly, silty sand over glacial till. Boulder concentrations are prevalent throughout the area.

(3) Borrow Sources.- Ample quantities of compact silty till suitable for impervious fill occur in the east bank of Beards Brook. The area selected for development is approximately 1000 ft. upstream of the dam and at an elevation which will allow a downhill haul to the embankment. Large quantities of pervious glacial outwash materials are located within a radius of two miles of the damsite. These deposits contain ample material for the pervious sections but do not contain sufficient coarse material to be suitable for coarse concrete aggregate. Exploration is continuing to determine a suitable source of aggregate.

d. Construction Material.- (1) Materials Required.- Principal materials required for construction of the project are summarized in the following tabulation:

Item	Quantity C.Y. (Emb. Meas.)
Impervious Fill	290,000
Random Fill	280,000
Pervious Fill	460,000
Selected Gravel	115,000
Concrete Aggregate and Special Gravel	35,000
Rock for Slope Protection	200,000

(2) Materials Available in Required Excavation.- Materials obtained from required excavations will be used for embankment purposes where suitable. Of the total excavation, approximately 396,000 cubic yards of material is suitable for use in the embankment for the following purposes:

Item	Quantity C.Y. (Exc. Meas.)
Random Fill and Backfill	210,000
Rock for Slope Protection	140,000
Semi-compacted Fill	46,000

Rock for slope protection is available from rock excavation at the spillway and from boulders encountered in required excavations.

(3) Materials Available for Borrow.- (a) Impervious Borrow.- Massive deposits of sandy and silty glacial till form the walls of the present stream valley. These deposits are excellent sources of material for impervious embankment construction. The area selected for impervious borrow is on the east bank of Beards Brook, a short distance upstream from the dam site. The material selected for use is the unweathered, unmodified till which occurs under a surface cover of gravelly sand. Boulders are densely strewn over the ground surface in this locality.

(b) Pervious Borrow.- Large quantities of glacio-fluvial materials occur a short distance upstream from the dam site. They are outwash deposits and are composed principally of coarse to fine sand. The area selected for development is approximately 1.5 miles upstream from the dam site (Plate II-7).

(c) Processed Aggregates.- 1. Investigation of existing commercial sources of processed aggregate for concrete and special gravels has been made. Reports have been compiled covering plant facilities, materials processed and experience records. Samples of materials from principal sources have been obtained for classification and laboratory testing. Investigations to date indicate that suitable materials exist within reasonable hauling distance. The nearest existing commercial plants are located in Manchester and Keene, N.H., each approximately 30 miles from the project location.

2. Investigation of an undeveloped source of aggregate within ten miles of this project has been made. The possibilities of developing this source for use in connection with other projects in the vicinity are being considered.

(d) Rock for Slope Protection.- Rock for slope protection may be obtained from rock excavation for spillway construction, or from a combination of this rock excavation and of boulders in required excavations. Bedrock excavation at the spillway, and boulders in earth excavation are composed substantially of hard durable granite.

(4) Materials Summary.- A summary of construction materials required with indicated source, and materials available from required excavation and selected borrow areas, with indicated disposition, is shown in the following tabulation.

CONSTRUCTION MATERIALS

MATERIALS REQUIRED FOR EMBANKMENT

ITEM	QUANTITY – C.Y. Embankment Measure	QUANTITY – C.Y. Excavation Measure	Source Excavation Measure C.Y.	
Compacted Impervious Fill	290,000	335,000	335,000	Impervious Borrow Area
Compacted Random Fill	280,000	320,000	200,000	Structure Excavation
			70,000	Impervious Borrow Area
			50,000	Pervious Borrow Area
Semi-compacted Fill	40,000	46,000	46,000	Foundation Stripping
Compacted Pervious Fill	460,000	530,000	530,000	Pervious Borrow Area
Selected Gravel	115,000	130,000	130,000	To be selected from available sources
Processed Aggregates and Special Gravels	35,000	50,000	50,000	Processing Plant
Rock for Slope Protection	200,000	140,000	75,000	Structure Excavation (Rock)
			50,000	Structure Excavation (Cobbles & Boulders)
			15,000	Boulders from Stripping
Topsoil & Organic Subsoil	10,000	10,000	10,000	Stripping

CONSTRUCTION MATERIALS

MATERIALS AVAILABLE FROM EXCAVATION AND BORROW

ITEM	QUANTITY – C.Y.	Disposition Excavation Measure C.Y.
Stripping Foundation	145,000	10,000 Topsoil + Organic Subsoil
		15,000 Cobbles and boulders for riprap
		46,000 Semi-compacted Fill
		74,000 Waste
Structure Excavation Earth	260,000	10,000 Structure Backfill
		50,000 Cobbles and boulders for riprap
		200,000 Compacted Random Fill
Rock	75,000	75,000 Rock for Slope Protection
Impervious Borrow	450,000	335,000 Compacted Impervious Fill
		70,000 Compacted Random Fill
		45,000 Stripping and Waste
Pervious Borrow	640,000	530,000 Compacted Pervious Fill
		50,000 Compacted Random Fill
		60,000 Stripping Waste

e. Soil Data.- (1) Scope and Extent of Laboratory Investigation.- All samples of material encountered in field exploration have been submitted to the laboratory for final classification and testing. Selected representative samples have been tested to determine compaction characteristics, shear strength, permeability and consolidation characteristics, Investigation of these soil properties is continuing but sufficient data have been obtained to determine the general range of results.

(2) Laboratory Procedures.- (a) Mechanical Analysis.- Mechanical analyses of selected representative samples have been made using a standard sieve analysis with a minimum sieve of 100 meshes per inch, and hydrometer analysis of materials passing that sieve size.

(b) Specific Gravity.- The specific gravity of principal materials has been obtained by the water displacement method (A.A.S.H.O. T100-38).

(c) Density.- Density of cohesive soils has been determined from undistrubed paraffined chunk samples. Density of cohesionless soils has been determined in the field by the sand displacement method.

(d) Water Content.- The natural water content of principal materials was determined from samples obtained in the field and transported to the laboratory in sealed jars. Results are reported in terms of percentage of oven dry weight.

(e) Compaction Characteristics.- Compaction characteristics of cohesive soils are determined from the moisture density relation obtained using standard and modified Proctor Compaction Tests. Compaction characteristics for cohesionless soils include determination of minimum dry density by placing soil in a container in the loosest possible condition without vibration or impact, and maximum dry density obtained by combined pressure and vibration. Identification of the state of compaction of cohesionless soils is made by reference to the density ratio D_r, as defined by the expression

$$D_r = \frac{d - d_o}{d_{100_v} - d_o}$$

where d = dry density, p.c.f., of soil being considered.

d_o = dry density, p.c.f., of soil in loosest state from laboratory test for minimum density, placed dry.

d_{100_V} = dry density, p.c.f., of soil in densest state from laboratory test for maximum density, placed dry and compacted under combined influence of pressure and vibration by method developed in the Providence District Soils Laboratory.

(f) Shear Strength.- Shear strength of principal materials was determined using a triaxial compression device in accordance with procedures outlined by A. Casagrande and R. E. Fadum, in "Notes on Soil Testing for Engineering Purposes", a publication from Harvard University Graduate School of Engineering.

(g) Permeability.- Permeability of materials was determined using de-aired water in a falling head type apparatus with a plastic, transparent permeameter following the general procedure described by G. E. Bertram in "An Experimental Investigation of Protective Filters", a publication of Harvard University Graduate School of Engineering.

(h) Consolidation.- Laboratory consolidation characteristics are determined by using fixed ring consolidation test apparatus for a 4-1/4 inch diameter sample of 1-1/4 inch in initial thickness.

(3) Test Results.-(a) Classification of Materials Encountered.- Classification of materials encountered in field explorations are shown by graphic logs of explorations on Plates II-4, II-5, and II-8. This classification includes color, compactness, consistency, plasticity and basic soil type of each stratum encountered.

(b) Soil Data Summary.- 1. Results of laboratory tests performed on samples of principal foundation and borrow soils are summarized on Plates II-9 and II-10.

2. On Plate II-9, Figures 1, 2 and 3 show gradations of principal soils, and Figure 4 is a summary of properties of materials encountered in required excavations and of materials required for embankment. Conservative estimated values shown in Figure 4 are based on results of these tests augmented by tests on similar materials for other sites.

3. Plate II-10 shows comparative plots of tests for compaction characteristics, shear strength and permeability of principal soils. Data are arranged from left to right in order of increasing permeability of materials tested.

f. Embankment Design.- (1) Design Criteria.- The embankment design involves a study of foundation conditions, the determination of the characteristics of foundation and borrow materials, the choice of a section which utilizes economically the available materials and which is safe under all conditions. The embankment design must satisfy the following criteria:

(a) The slopes of the embankment must be of such that no shear slide can occur in the embankment or foundation materials.

(b) The void ratio of all materials in which a flow slide might occur must be less than the critical void ratio.

(c) Seepage must be controlled so that no detrimental uplift pressures or transportation of material can occur.

(d) Provision should be made to compensate for the settlement of the embankment after construction to insure the design free board height.

(2) Preliminary Embankment Design.- Several preliminary project designs have been considered for various alignments within the general project area. The embankment design presented for review by the Board of Consultants in December 1944 was essentially the same as reported herein with the following exceptions:

(a) Size of the compacted random section has been decreased and corresponding increases have been made in the compacted random and compacted pervious sections.

(b) The downstream slope protection above elevation 640 has been changed from seeded topsoil to dumped rock. This change eliminates the need for the berms originally planned.

(3) Definite Project Design Features.- (a) Foundation Conditions.- Foundation conditions have been described for four different portions of the dam site (Paragraph b).

(b) Compacted Impervious Section.- The compacted impervious section of the embankment consists of a central core of till which contacts till in the foundation from the east abutment to contact with bedrock at the east side of the spillway wall in the west abutment.

(c) Compacted Random Sections.- The compacted random sections of the embankment have been included in the design to provide a transition between the impervious core and the pervious section of the dam. Dimensions of these random sections have been chosen to utilize the estimated quantity of suitable material from structure excavation at the dam site and from the selected borrow areas.

(d) Compacted Pervious Sections.- The compacted pervious sections of the dam have been designed to provide sufficient stability of the upstream section during rapid drawdown of water elevation and to hold the maximum line of seepage in the downstream sections well below ground surface for conditions of sustained high upstream pool elevation.

(e) Downstream Toe Drain.- Water seeping through and under the dam is collected in a downstream toe drain of very pervious gravel containing a perforated collector pipe connected to outlet at convenient intervals. In the major valley section the toe drain material extends upstream as a drainage blanket under the compacted pervious section to 10 feet from the downstream random section.

(f) Control of Subsurface Water Pressure.- Preliminary exploratory drilling encountered artesian flow from a stratum of sand under the till approximately 60 feet below ground surface in the valley bottom of the Beards Brook channel. Exploration and observations are continuing to investigate more fully the existing conditions, and to provide the basis for design of a suitable method of control by use of drain wells.

(g) Slope Protection.- Both the upstream and downstream slopes of the embankment are protected by a blanket of dumped stone on a filter layer of screened gravel.

(4) Design Studies.- (a) Seepage Analysis.- The seepage through and under the embankment has been studied by flow nets (Plate II-11). As the impervious core of the dam contacts a substantial body of equally impervious till throughout the entire length of the embankment very little seepage is anticipated. The pervious sand underlying till in the bottom of the Beards Brook Valley is well blanketed by till and has little influence on underseepage. Based on the flow net studies the total seepage through and under the embankment is estimated at 0.1 c.f.s. for sustained maximum upstream water surface.

(b) Filter Analysis.- In all sections of the embankment and its foundation through which water passes, a study has been made to insure that the gradation of adjacent soils is such that the finer sizes of one soil will not be transported by the seeping water into the voids of an adjoining soil. In areas where seepage water is collected, the gradations of adjacent soils fulfill the above criteria and in addition the soils become progressively several times more pervious in the direction of discharge. The criteria used for these analyses are those contained in Chapter XXI of the Military Engineering Manual, OCE. The determination of the general range of material for the filter blanket in the downstream section of the earth embankment is illustrated on Plate II-11.

(c) Embankment Stability.- 1. Method of Analysis.- Using the most dangerous circle method, the stability ratio for shear failure of the dam embankment and its foundation was determined by investigating the forces tending to cause movement and those producing potential resistance to movement on several circular sliding surfaces of weakness which were selected by systematic trial. This method investigates only the possibility of a shear failure. The analysis of flow slide failure is described in a following subparagraph. In the analysis the driving forces include the rotating effect of the weight of the soil mass and water above the surface of failure and also the forces generated by water pressure. The forces producing potential resistance consist of the shear strength generated along the sliding surface. The ratio of the potential resisting force and the driving force is termed the stability ratio. A sufficient number of potential surfaces were analyzed to determine the position of the surface having the least stability ratio, termed the "minimum stability ratio". A minimum stability ratio of unity indicates equality of driving and potential resisting forces and implies that the embankment is on the verge of failure, while a minimum stability ratio of greater than unity indicates that the structure possesses reserve strength.

2. Sections Analyzed.- Analyses were made of the typical deep valley section and a representative section above elevation 680 (Plate II-12). In the analysis the embankment and foundation have been considered as an integral mass. The soil characteristics used in the analyses of the various sections are shown on Plates II-9 and II-10.

3. Analysis Results.- Results of several stability trials of both upstream and downstream slopes are summarized in the following tabulation:

Section	Minimum Stability Ratio	
	Upstream	Downstream
Typical Deep Valley	2.0	2.2
Typical above El. 680	2.1	2.5

4. Foundations and Abutments.- Stability of the foundation and abutment in analyses for possible shear failure was investigated in conjunction with analyses of the upstream and downstream slopes. The abutment and minor portions of the embankment are considered of greater stability than the principal embankment section for which the analysis is shown on Plate II-12, the results of which are tabulated in paragraph 3 above.

(d) Flow Failure Analyses.- A flow failure is defined as the liquefaction by shock of a mass of loose, saturated cohesionless material. Based upon the experience gained by the detailed study of flow failure made in connection with the design of the Franklin Falls Dam, a flow failure of either the upstream compacted pervious or random sections of the embankment or the embankment foundation is considered highly improbable. An analysis of the possibility of a flow failure will be made in connection with the final design.

(e) Surface Slides.- Surface slides may occur during the period of frost melting in soils affected by frost action. Such surface slides will not occur in the embankment since all materials, in the range of frost penetration (at this site about four feet) are cohesionless and not susceptible to frost action.

(f) Settlement Analysis.- Based on experience gained as a result of settlement observations on the completed Franklin Falls Dam, a settlement of the embankment at its maximum section is expected to be approximately 1 to 2 inches due to foundation consolidation and 2 to 4 inches due to consolidation of the compacted impervious section under its own weight. Total settlement will occur gradually over a period of several years. Upon further analysis and before final design plans are prepared, definite values will be established for constructing the embankment to sufficient initial height to allow for this settlement.

g. Outlet Works Foundation.- (1) Location.- The outlet works are located at the easterly side of the existing channel of Boards Brook and under the major embankment section.

(2) Foundation Conditions.- The foundation for the outlet works consists of the very compact glacial till of the massive deposit extending across the entire valley. At a depth of more than 60 feet a deposit of pre-glacial sand occurs over bedrock (Plate II-6).

(3) Foundation Design.- The existing foundation soil possesses ample strength to support the load of the outlet works and embankment. Attention has been given to design details to obtain adequate protection against flowing water and frost action (Plates IV-3 and IV-4).

h. Spillway Foundation.- The spillway is located in bedrock excavation in the existing North Branch Brook channel and presents no design problem. This rock excavation is suitable for embankment slope protection.

i. Diversion Channel and Canal.- The diversion channel and canal connect the North Branch Brook valley and the Beards Brook valley in the vicinity immediately upstream from the embankment. Foundation conditions have been described in Paragraph c. Design details include protection of structure against damage by flowing water and frost action.

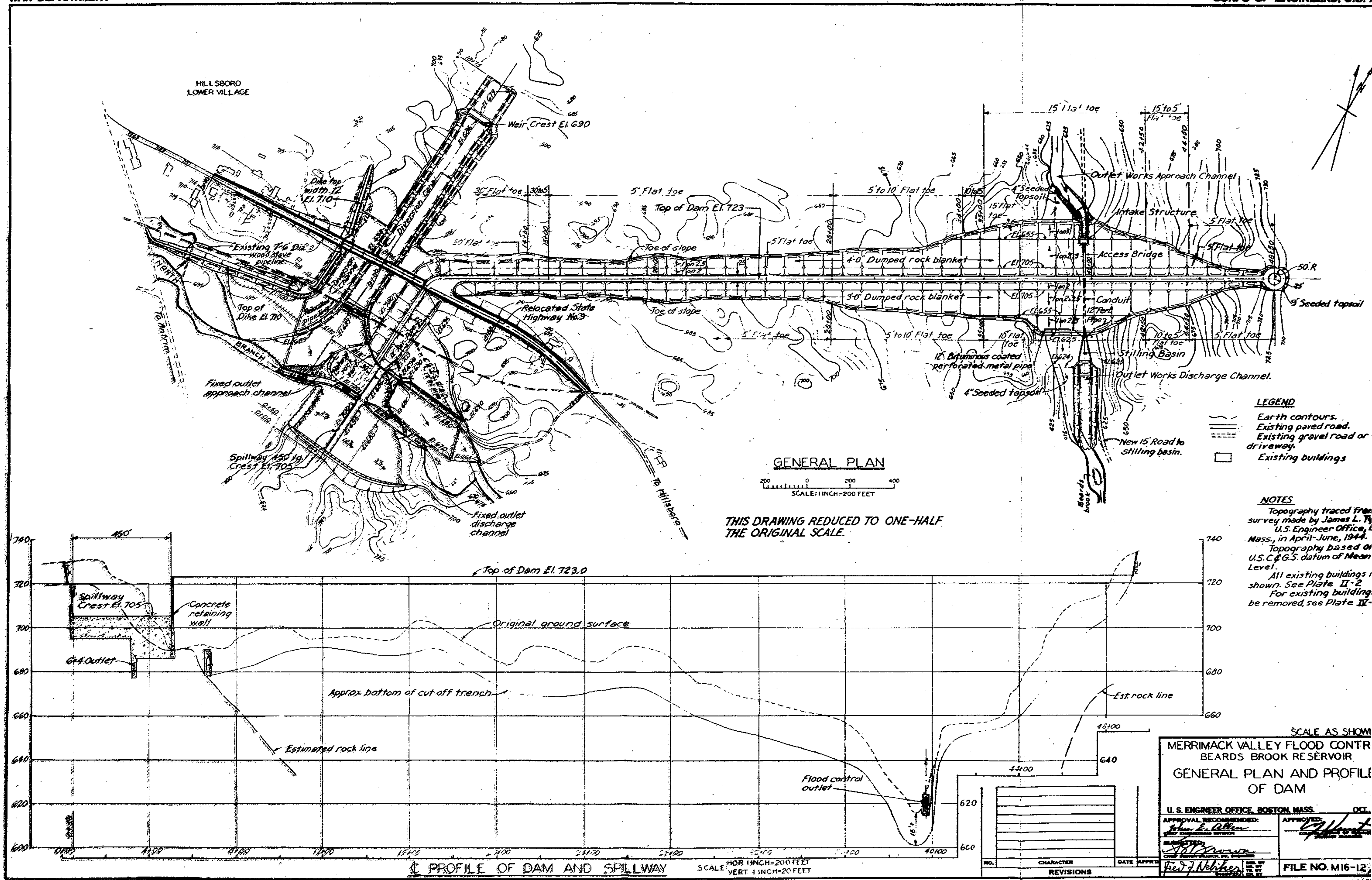
j. Construction Procedure.- (1) Sequence of Operations.- The design contemplates a 3-season construction schedule. During the first season, it is proposed to perform all foundation stripping, to install the relocated penstock, to excavate the diversion channel and canal, and to construct part of the spillway and embankment. During the second season, it is proposed to complete all work except completion of the access roads and final grading.

(2) Cofferdams.- The principal cofferdam occurs as part of the embankment across Beards Brook valley (Plate IV-3). Other minor cofferdams will be required in other parts of the work for only very short periods of time.

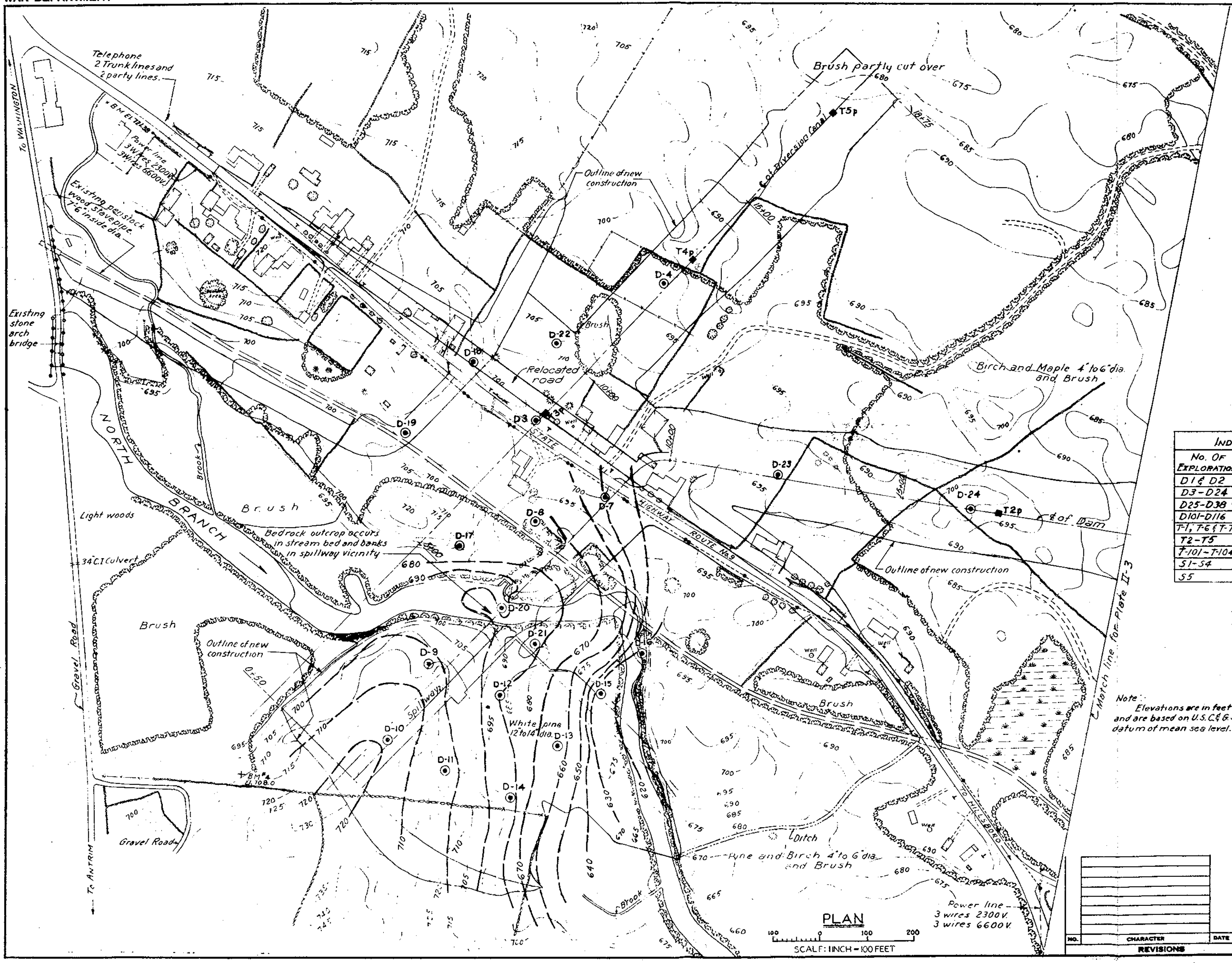
(3) Embankment Construction.- Construction of the compacted earth embankment requires the simultaneous placement of the compacted impervious, random and pervious sections in parallel layers for the full width of the dam with the surface sloped to drain readily at all times.

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S.



WAR DEPARTMENT CORPS OF ENGINEERS, U.S. AR



INDEX FOR SUBSURFACE EXPLORATION			
No. OF EXPLORATION	PURPOSE	PLAN PLATE No.	GRAPHIC PLATE
D1 & D2	Foundation	II-3	II-4
D3-D24	Foundation	II-2	II-4
D25-D38	Foundation	II-3	II 5
D101-D116	Pervious borrow	II-7	II-8
T-1, T-6 & T-7	Foundation	II-3	II-5
T2-T5	Foundation	II-2	II-5
T-101-T-104	Borrow	II 7	II-8
S1-S4	Impervious borrow	II-7	II-8
S5	Foundation	II-3	II-3

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALF

Note: Elevations are in feet and are based on U.S.C.&G.S. datum of mean sea level.

LEGEND

Existing stone wall
Existing fence
Earth contours
Rock contours
Existing dirt road
Rock outcrop
D-24 Drill hole
T4p Test pit
(o.w) Observation well.
B S-5 A Seismic exploration.

SCALE AS SHOWN

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTRO
BEARDS BROOK RESERVOIR
PLAN OF FOUNDATION
EXPLORATION NO. 1

U. S. ENGINEER OFFICE, BOSTON, MASS. OCT., 194

APPROVAL RECOMMENDED: John E. Allen
CHIEF ENGINEERING DIVISION

SUBMITTED:
CHIEF DESIGN BRANCH, PR. ENGINEER

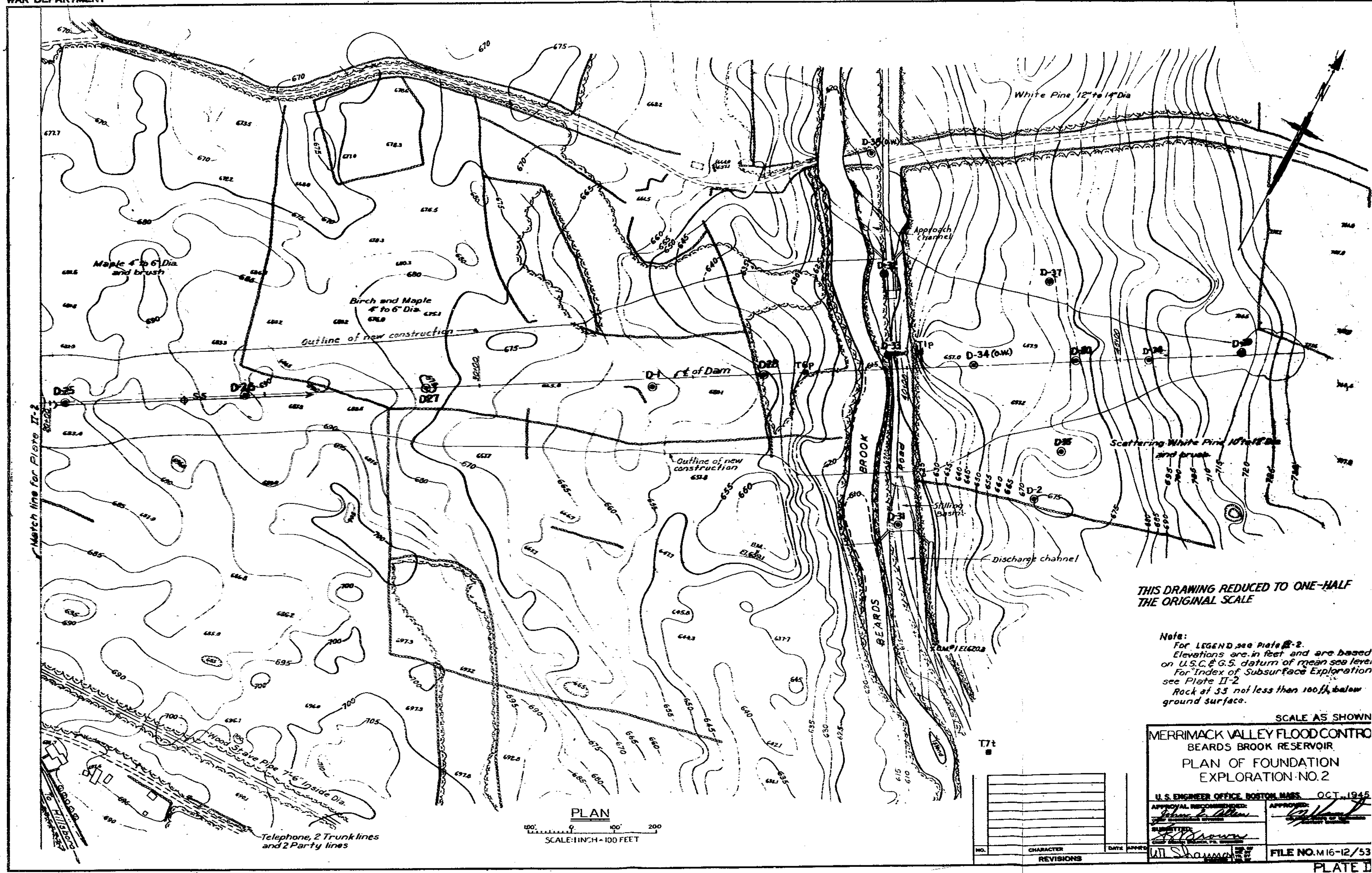
APPROVED:
DISTRICT ENGINEER

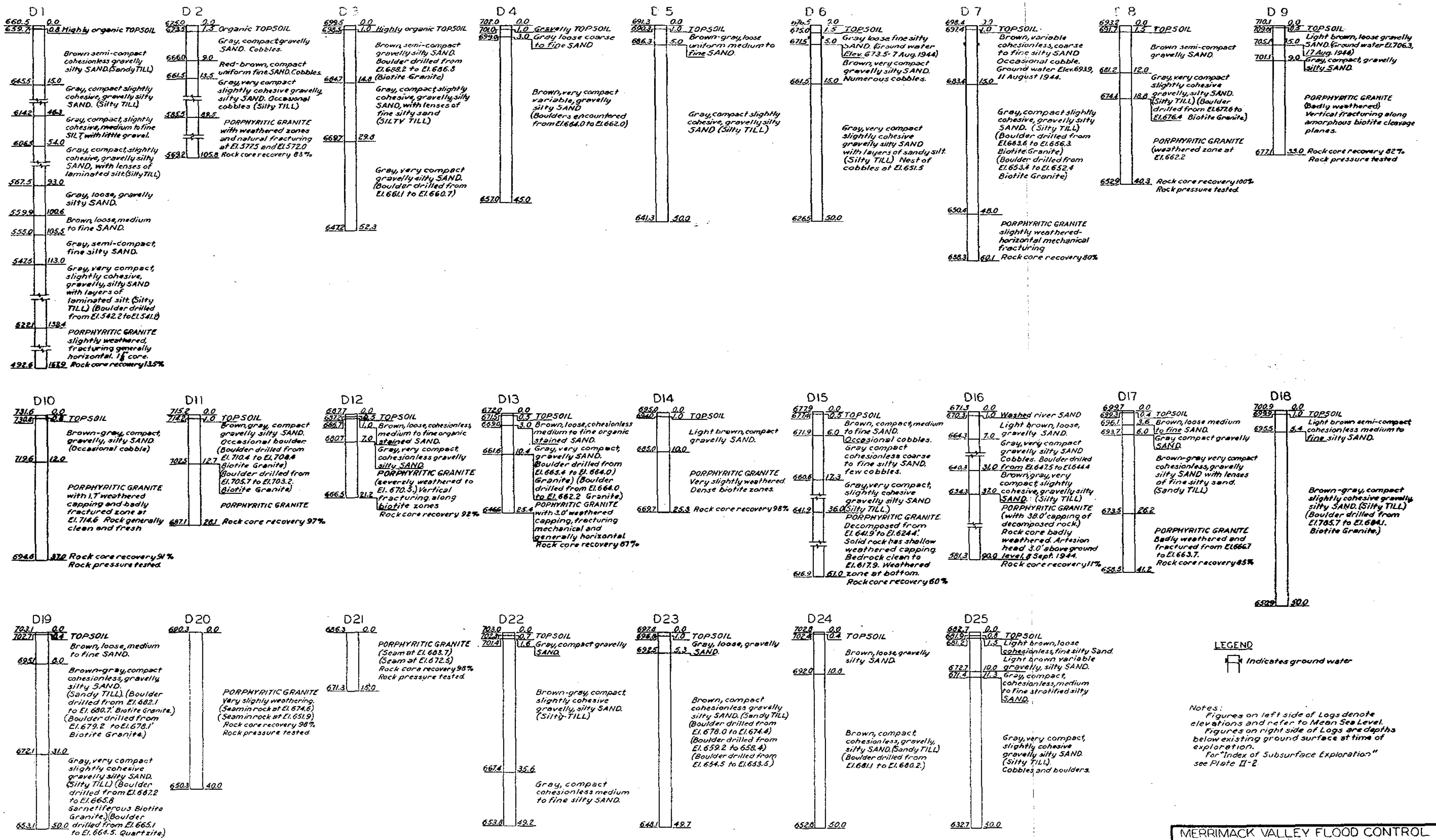
FILE NO. M16-12/5

PLATE I

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARM





Notes:
Figures on left side of Logs denote elevations and refer to Mean Sea Level.
Figures on right side of Logs are depths below existing ground surface at time of exploration.
For "Index of Subsurface Exploration" see Plate II-2

NO.	CHARACTER	DATE	BY
	REVISIONS		

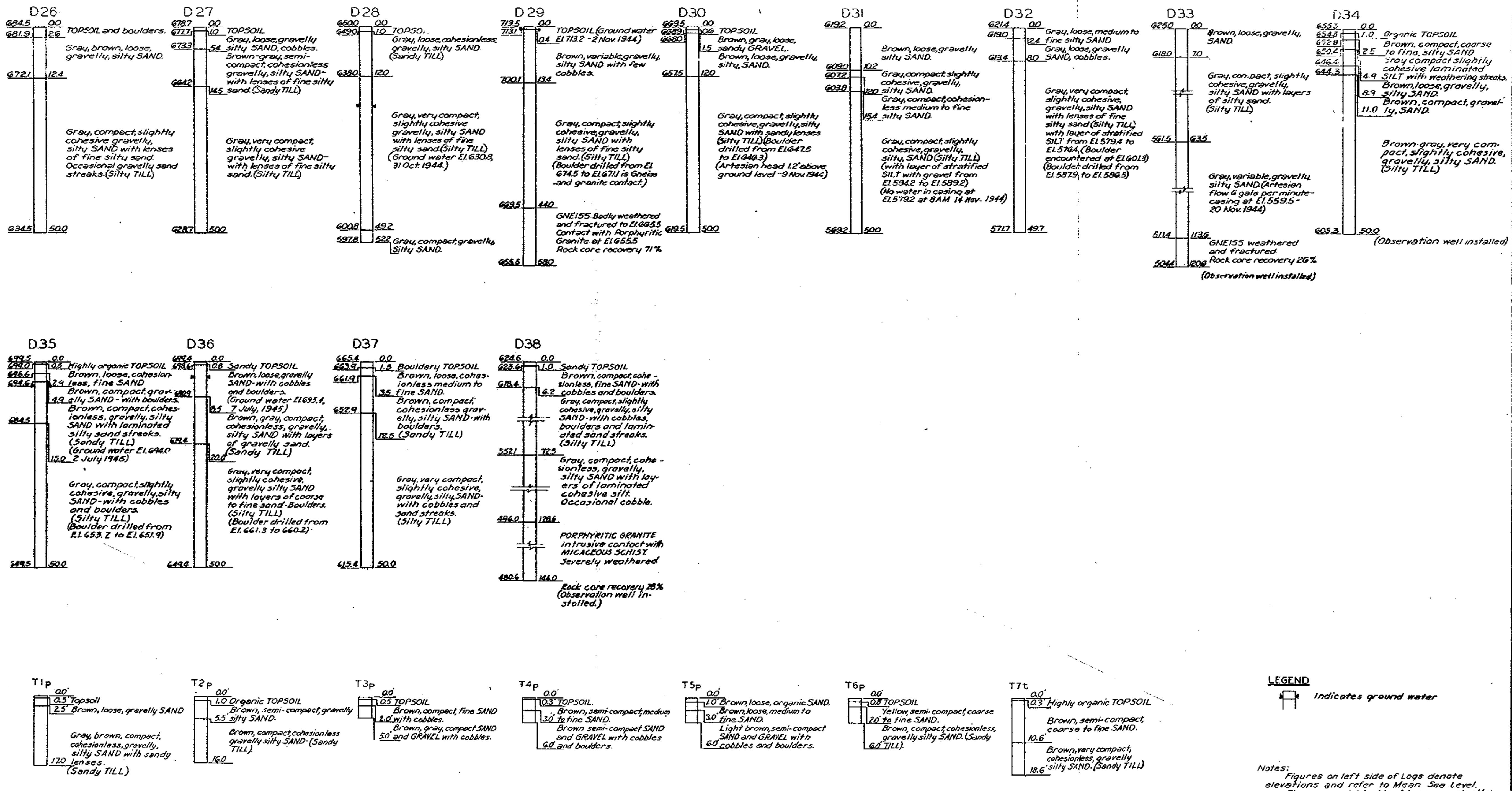
MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
RECORD OF FOUNDATION EXPLORATION NO. 1

U. S. ENGINEER OFFICE, BOSTON, MASS. OCT., 1945

APPROVAL RECOMMENDED: John E. Allen
APPROVED:
SUBMITTED:
W. H. Shannon

FILE NO. M16-12/54

PLATE II-4



LEGEND

Indicates ground water

Notes:
Figures on left side of Logs denote elevations and refer to Mean Sea Level.
Figures on right side of Logs are depths below existing ground surface at time of exploration.
For "Index of Subsurface Exploration" see Plate II-2
Artesian flow at D-33 & D-38 has been continuous since date of exploration.

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
RECORD OF FOUNDATION
EXPLORATION NO. 2

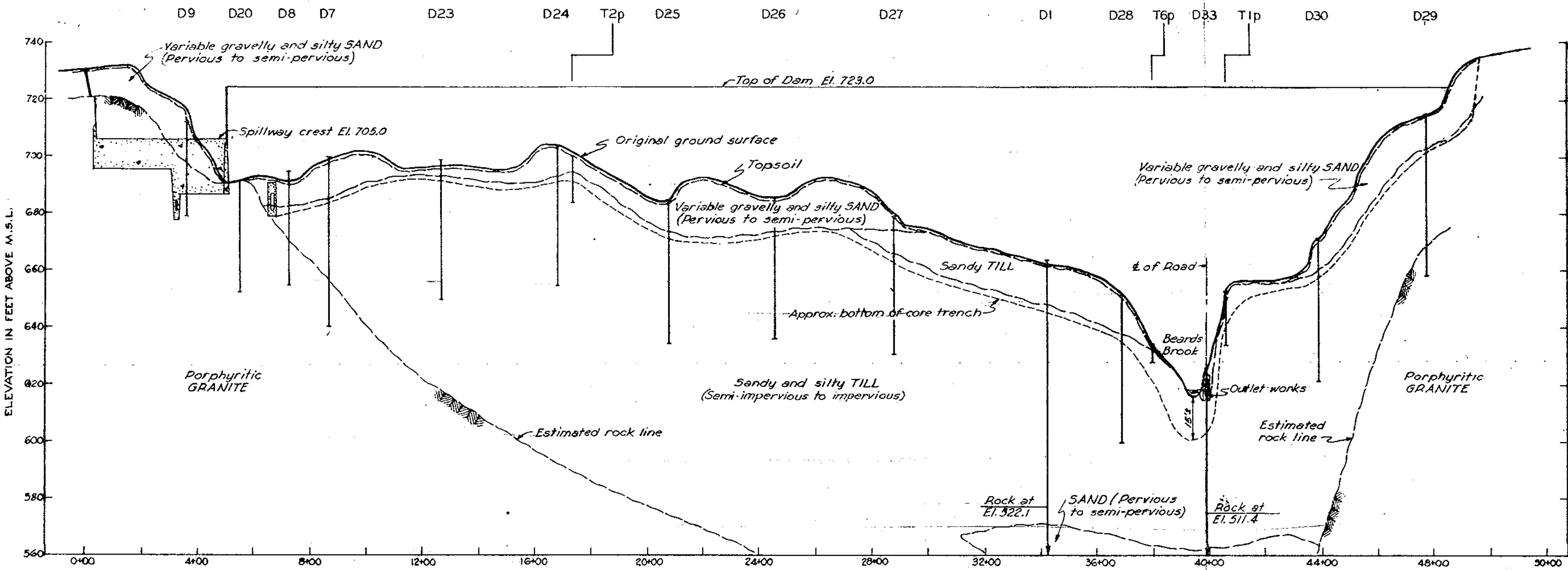
U. S. ENGINEER OFFICE, BOSTON, MASS. OCT., 1945

APPROVAL RECOMMENDED: John E. Allen
APPROVED:
SUBMITTED:

NO.	CHARACTER	DATE	BY
REVISIONS			

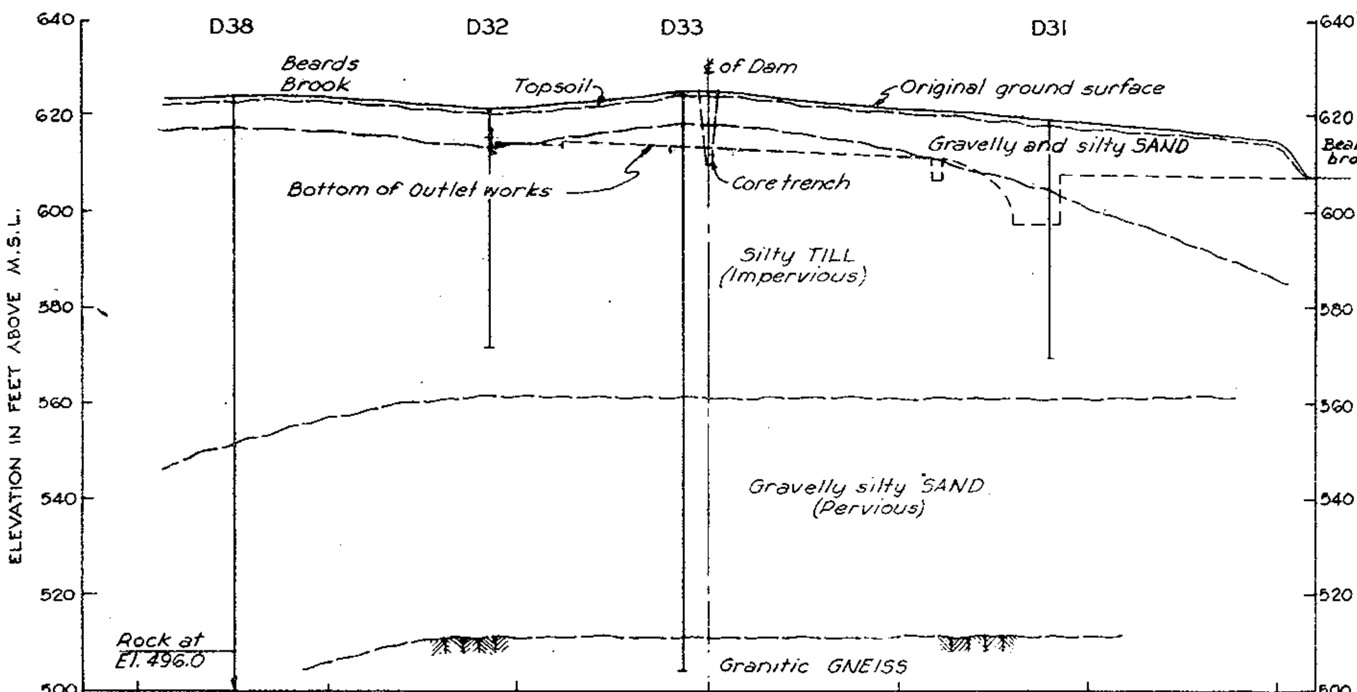
FILE NO M16-12/55

PLATE II-5



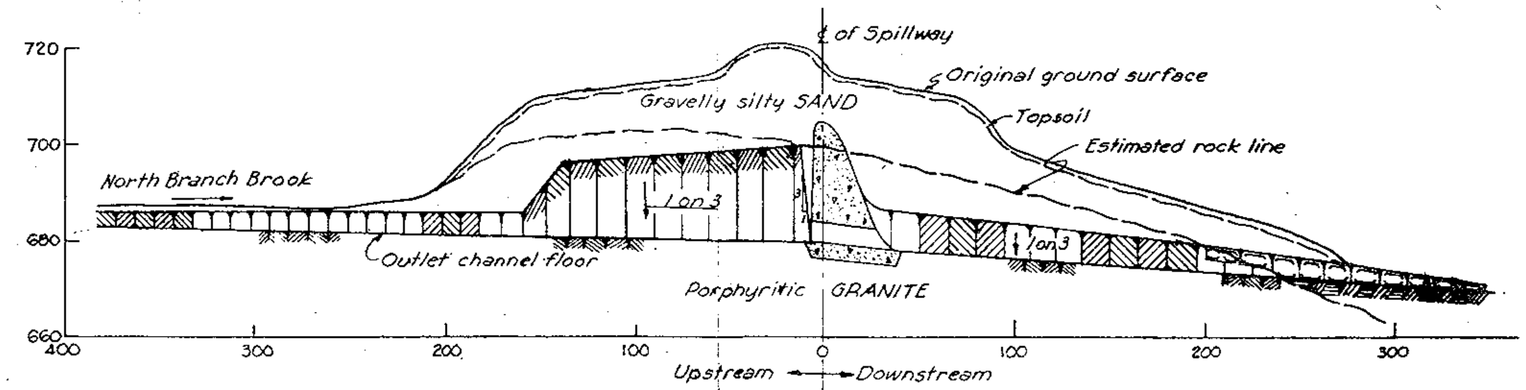
℄ PROFILE OF DAM AND SPILLWAY

SCALE { HOR. 1 INCH = 200 FEET; VERT. 1 INCH = 20 FEET



SECTION THRU OUTLET WORKS

SCALE { HOR. 1 INCH = 100 FEET; VERT. 1 INCH = 20 FEET



SECTION THRU SPILLWAY AND OUTLET FOR NORTH BRANCH

SCALE { HOR. 1 INCH = 50 FEET; VERT. 1 INCH = 20 FEET

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

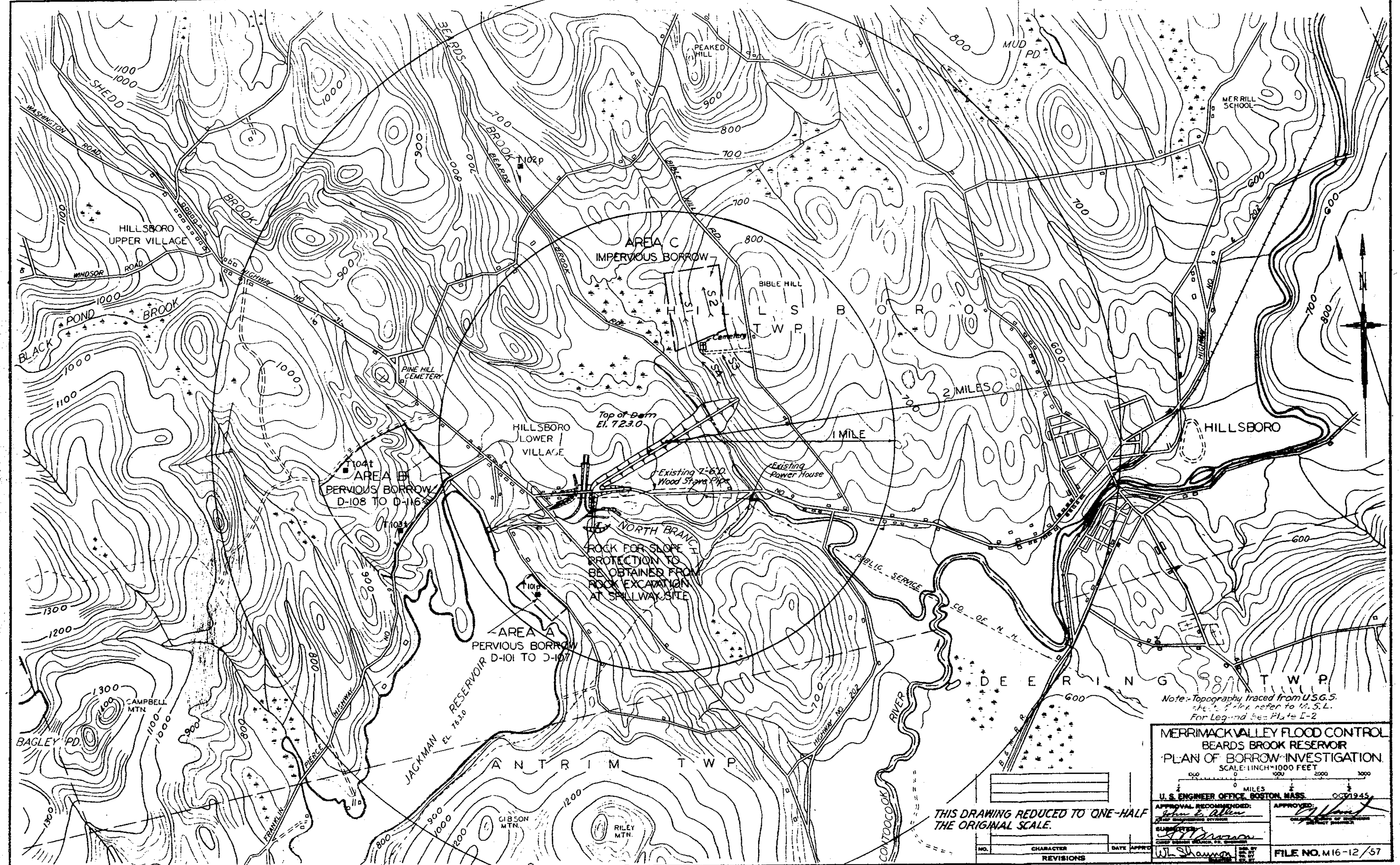
NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
GEOLOGICAL PROFILES
SCALE AS SHOWN

U.S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945

APPROVAL RECOMMENDED: John E. Allen
SUBMITTED: R. Brown
APPROVED:

FILE NO. M16-12/56


MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
PLAN OF BORROW INVESTIGATION
SCALE: 1 INCH = 1000 FEET
U.S. ENGINEER OFFICE, BOSTON, MASS. OCT 1945
FILE NO. M16-12/57
THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.
Note:- Topography traced from U.S.G.S. sheet. Elev. refer to M.S.L. For Legend See Plate I-2
AREA C
IMPERVIOUS BORROW
AREA B
PERVIOUS BORROW
D-108 TO D-116
AREA A
PERVIOUS BORROW
D-101 TO D-107
ROCK FOR SLOPE PROTECTION TO BE OBTAINED FROM ROCK EXCAVATION AT SPILLWAY SITE
Top of Dam El. 723.0
Existing 7'-6" D. Wood Stave Pipe
Existing Power House
1 MILE
2 MILES
HILLSBORO
HILLSBORO LOWER VILLAGE
HILLSBORO UPPER VILLAGE
HILLSBORO TWP.
DEERING TWP.
ANTRIM TWP.
BIBLE HILL
PEAKED HILL
MUD PD.
MERRILL SCHOOL
PINE HILL CEMETERY
Cemetery
NORTH BRANCH
BEARDS BROOK
SHEDD BROOK
BLACK POND BROOK
WASHINGTON ROAD
WINDSOR ROAD
BIBLE HILL RD.
HIGHWAY NO. 31
HIGHWAY NO. 9
HIGHWAY NO. 202
PIERCE HIGHWAY
FRANKLIN
PUBLIC SERVICE CO. OF N.H.
JACKMAN RESERVOIR EL. 763.0
CONTOOCOOK RIVER
CAMPBELL MTN.
BAGLEY PD.
GIBSON MTN.
RILEY MTN.
B. & M. R.R.
APPROVAL RECOMMENDED:
APPROVED:
SUBMITTED:
REVISIONS
NO.
CHARACTER
DATE

D101
- 0.0'–1.0' TOPSOIL
- Light brown, compact, fine silty SAND.
- 11.0' Brown, compact, gravelly, SAND.
- 16.0' Brown, compact, cohesionless, gravelly, silty SAND.
- 17.0' (Sandy TILL.)

D102
- 0.0'–1.0' TOPSOIL
- Brown, compact, gravelly, SAND
- 10.0' Brown, semi-compact, SAND and GRAVEL
- 15.0' Brown, compact, gravelly SAND with cobbles.
- 19.0' Brown semi-compact SAND and GRAVEL. Numerous cobbles and boulders.
- 23.0'

D103
- 0.0'–1.0' TOPSOIL
- Brown, semi-compact gravelly SAND. (Numerous boulders.)
- 10.0' Brown, very compact, cohesionless, gravelly silty SAND.
- 14.0' (Sandy TILL.)

D104
- 0.0 Brown, loose, gravelly, SAND.
- 9.0' Brown, compact, cohesionless, fine SAND with cobbles.
- 15.0' Brown, compact, cohesionless, gravelly, silty SAND.
- 18.0' Sandy TILL.

D105
- 0.0'–1.0' TOPSOIL
- Brown, loose, gravelly SAND.
- 10.0' Brown, compact, cohesionless, medium to fine, SAND. (Occasional cobble)
- 16.0' Brown, compact, cohesionless, gravelly, silty, SAND.
- 20.0' (Sandy TILL.)

D106
- 0.0'–1.0' TOPSOIL.
- Brown, compact, gravelly, SAND.
- 9.0' Brown, compact, cohesionless, gravelly, silty, SAND.
- 11.0' (Sandy TILL.)

D107
- 0.0' Brown, semi-compact, cohesionless, medium to fine SAND.
- 4.0' Refusal.

D108
- 0.0'–0.5' TOPSOIL
- Brown, compact, gravelly SAND.
- 40.0'

D109
- 0.0'–0.5' TOPSOIL
- Brown, compact, gravelly, SAND.
- 21.0' Brown, compact, cohesionless, gravelly, silty, SAND.
- 25.0' (Sandy TILL.)

D110
- 0.0'–0.5' TOPSOIL
- Brown, loose, cohesionless, fine SAND.
- 2.0' Brown, semi-compact, gravelly, SAND.
- 10.0' Brown, compact, cohesionless, gravelly, silty SAND.
- 13.0' (Sandy TILL.)

D111
- 0.0'–0.5' TOPSOIL
- Brown, loose, cohesionless, medium to fine, SAND.
- 2.5' Brown, compact, gravelly SAND.
- 25.0' Brown, compact, cohesionless, gravelly, silty, SAND.
- 29.0' (Sandy TILL.)

D112
- 0.0' Brown, compact, gravelly, SAND.
- 41.0'

D113
- 0.0' Brown, compact, gravelly, SAND.
- 40.0'

D114
- 0.0'–1.5' Organic TOPSOIL
- Brown, loose, gravelly, SAND.
- 6.5' Brown, semi-compact, cohesionless, fine silty SAND.
- 15.0'

D115
- 0.0'–0.5' TOPSOIL
- Brown, compact, gravelly, SAND.
- 26.0' Refusal.

D116
- 0.0' Brown, compact, gravelly, SAND.
- 16.0' Refusal.

T101p
- 0.0' Brown, loose, gravelly, SAND.
- 5.0'

T102p
- 0.0' Brown, loose, gravelly SAND.
- 5.0'

T103t
- 0.0'–0.3' Organic SAND
- Brown, loose, SAND and Gravel.
- 14.0' Light brown, loose, cohesionless, medium to fine SAND.
- 18.0' Refusal

T104t
- 0.0'–0.3' Organic SAND.
- Yellow, loose, cohesionless, coarse to fine, SAND.
- 40.0'

SEISMIC DATA

SHOT	SURFACE ELEV.	ROCK ELEV.
S-1	692.2	611
S-2	736.4	Below 682
S-3	758.2	Below 690
S-4	690.5	632

Note:
Figures on right side of logs are depths below existing ground surface at time of exploration
For "Index of Subsurface Exploration" see Plate II-2

NO.	CHARACTER	DATE	BY
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
RECORD OF BORROW EXPLORATION

U. S. ENGINEER OFFICE, BOSTON, MASS. OCT., 1945

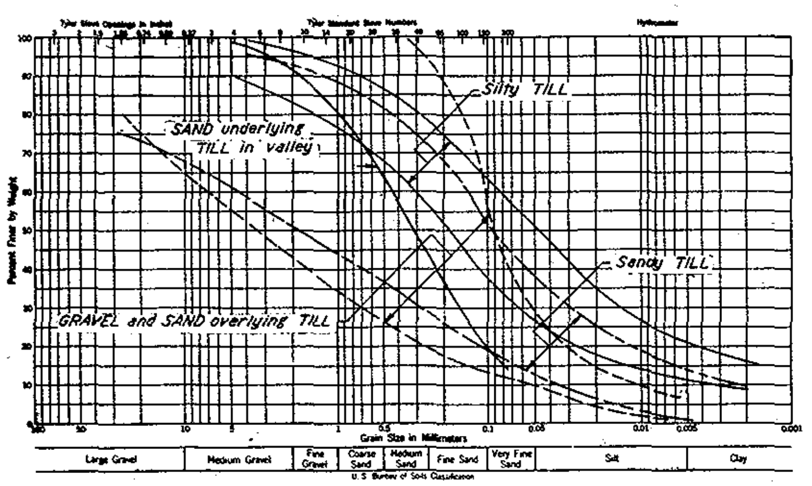
APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: [signature] COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: [signature] CHIEF DESIGN SECTION, ENGINEER

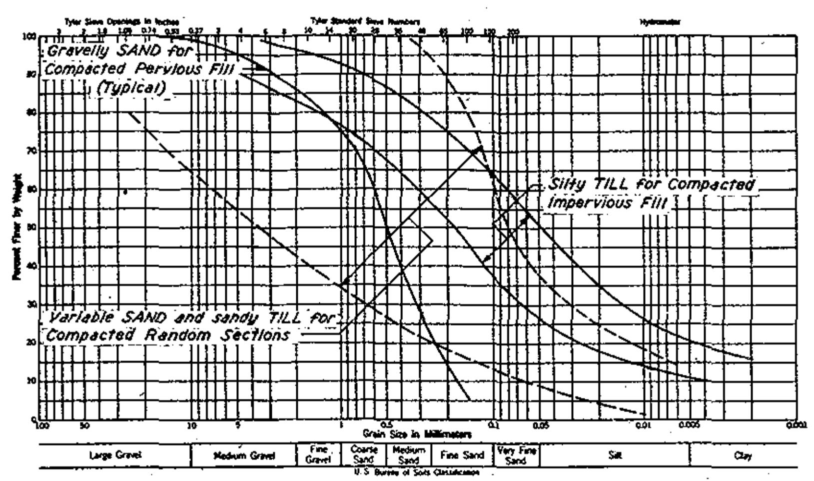
W. L. Shannon, ENGINEER — DES. BY / DR. BY / TR. BY / CK. BY

FILE NO. M16-12/58



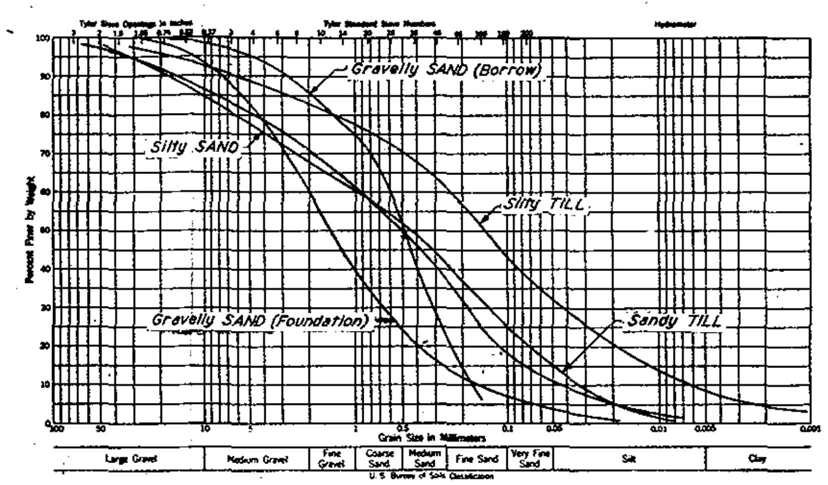
GRADATION RANGES OF FOUNDATION MATERIALS

FIG. 1



GRADATION RANGES OF EMBANKMENT MATERIALS

FIG. 2



GRADATION OF REPRESENTATIVE TEST SAMPLES

FIG. 3

IDENTIFICATION			NATURAL PROPERTIES					REMOLDED PROPERTIES					
								COMPACTION CHARACTERISTICS					
								LABORATORY TEST DATA					
TYPE	OCCURRENCE	CLASSIFICATION	WATER CONTENT w % dry wt.	DRY DENSITY d p.c.f.	VOID RATIO e	PERMEABILITY k 10^{-4} cm/sec.	SHEAR STRENGTH ϕ deg.	MIN. DRY DENSITY d p.c.f.	MAX. DRY DENSITY d p.c.f.	OPT. WATER CONTENT w % dry wt.	TOTAL SAMPLE MAX. DRY DENSITY d p.c.f.	PERMEABILITY k 10^{-4} cm/sec.	SHEAR STRENGTH ϕ deg.
1	2	3	4	5	6	7	8	9	10	11	12	13	14
Foundation Materials	Unconsolidated Surface Deposits	Variable gravelly and silty SAND	9.3 (a) (7) 2.6-19.5	127.3 (3) 126-128	0.31 (3) 0.30-0.33	5.0 (b) est.	35 est.	99.7 (c) (3) 97-103	121.3 (d) (3) 118-125	---	127.3 (e) (3) 126-128	1-8 See II-10 Fig. 9	See II-10 Fig. 7
	Upper zone of TILL	Weathered brown sandy TILL	9.5 (6) 8.5-11.1	125.9 (6) 119-133	0.34 (6) 0.27-0.41	0.1 est.	35 est.	---	131.5 (f) (2) See II-10 Fig. 2 130-133	8.3 (2) See II-10 Fig. 2 8.2-8.3	135.8 (2) 135-136	0.03-0.70 See II-10 Fig. 9	See II-10 Fig. 6
	Principal TILL body	Gray compact silty TILL	8 est.	139 (1)	0.21 (1)	0.01 est.	35 est.	---	136.0 (f) See II-10 Fig. 1	7.5 See II-10 Fig. 1	139.2 (1)	.004-.009 See II-10	See II-10 Fig. 5
Embankment Materials	Impervious Fill (Borrow)	Silty TILL	8 est.	139 (1)	0.21 (1)	0.01 est.	35 est.	---	136.0 (f) See II-10 Fig. 1	7.5 See II-10 Fig. 1	139.2 (1)	.004-.009 See II-10 Fig. 9	See II-10 Fig. 5
	Random Fill (Structure Excavation)	Sandy TILL	9.5 (6) 8.5-11.1	125.9 (6) 119-133	0.34 (6) 0.27-0.41	0.1 est.	35 est.	---	131.5 (f) (2) See II-10 Fig. 2 130-133	8.3 (2) See II-10 Fig. 2 8.2-8.3	135.8 (2) 135-136	0.30-0.70 See II-10 Fig 9	See II-10 Fig. 6
	Random Fill (Structure Excavation)	Variable gravelly and silty SAND	9.3 (7) 2.6-24.0	127.3 (3) 126-127	0.31 (3) 0.30-0.33	5.0 est.	35 est.	99.7 (3) 97-103	121.3 (d) (3) 118-125	---	127.3 (3) 126-128	1-8 See II-10 Fig. 9	See II-10 Fig. 7
	Pervious Fill (Borrow)	Gravelly SAND	5.5 (2) 5.4-5.6	115.2 (3) 113-120	0.46 (3) 0.40-0.49	100-500 est.	35 est.	97.0 (1)	117.0 (d) (1)	---	118.4 (1)	100-500 See II-10 Fig. 9	See II-10 Fig. 8

NOTES

(a) Top figures indicate average data. Figures in ( ) indicate number of tests. Figures below ( ) indicate range of test results.
(b) "est" indicates estimated values base on tests on similar materials for other project sites.
(c) Minimum density obtained by placing dry material passing No. 4 sieve in container without compaction or vibration.
(d) Maximum density obtained by Vibrated Density Test.
(e) Total densities are based on laboratory compaction test results adjusted for coarser fraction of sample not included in test specimens.
(f) Modified Proctor Tests.

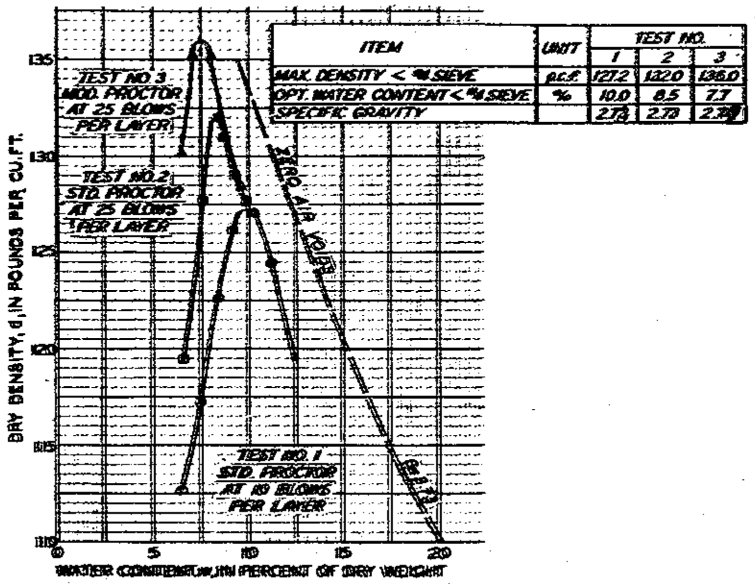
SUMMARY OF PROPERTIES OF MATERIALS ENCOUNTERED

FIG. 4

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR

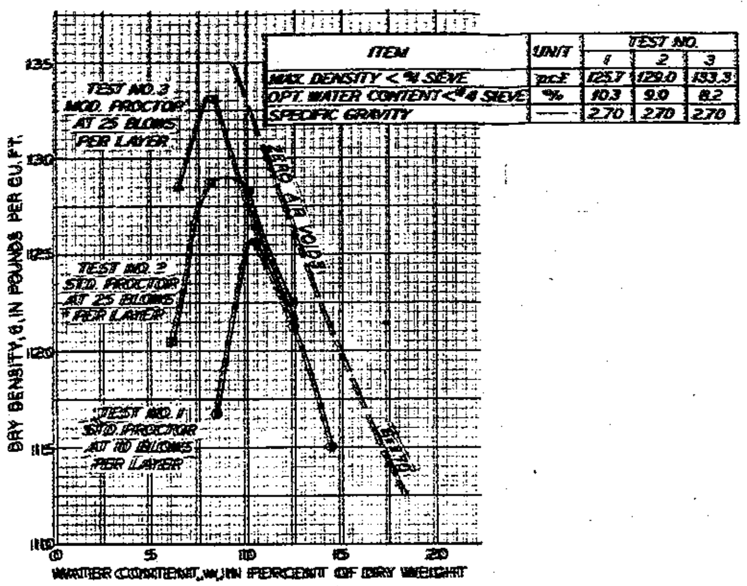
SOIL DATA SUMMARY NO. 1

FILE NO. M16-12/59
U.S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945



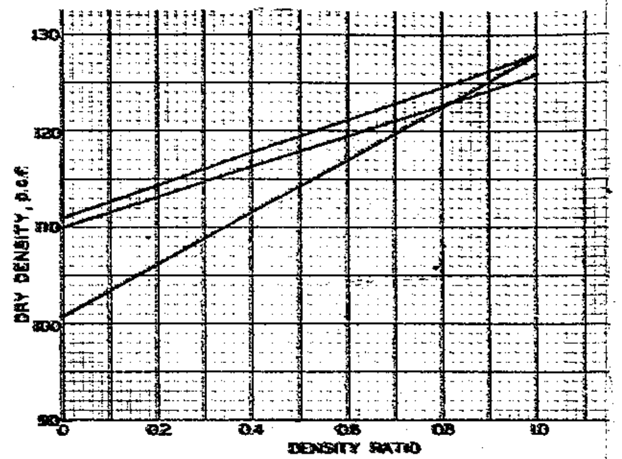
ITEM	UNIT	TEST NO. 1	2	3
MAX. DENSITY < #4 SIEVE	p.c.f.	127.2	132.0	136.0
OPT. WATER CONTENT < #4 SIEVE	%	10.0	8.5	7.7
SPECIFIC GRAVITY		2.73	2.73	2.73

MOISTURE-DENSITY PLOT FOR SILTY TILL
FIG. 1

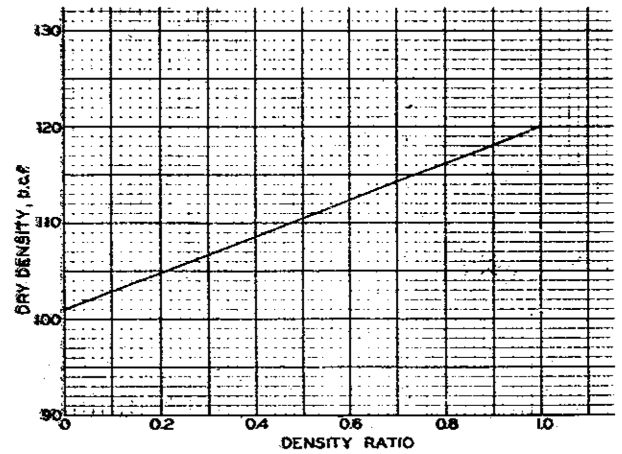


ITEM	UNIT	TEST NO. 1	2	3
MAX. DENSITY < #4 SIEVE	p.c.f.	125.7	129.0	133.3
OPT. WATER CONTENT < #4 SIEVE	%	10.3	9.9	8.2
SPECIFIC GRAVITY	—	2.70	2.70	2.70

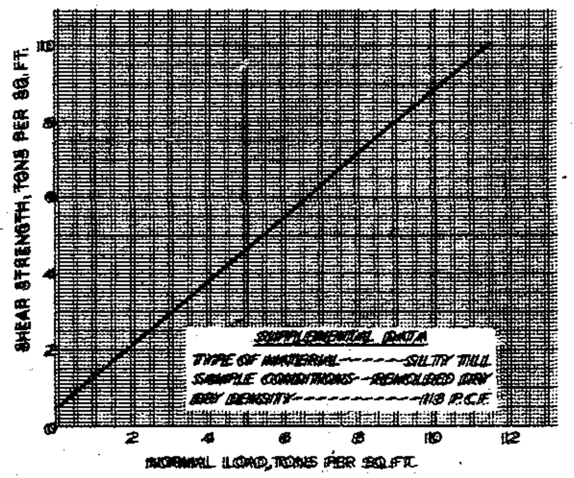
MOISTURE-DENSITY PLOT FOR SANDY TILL
FIG. 2



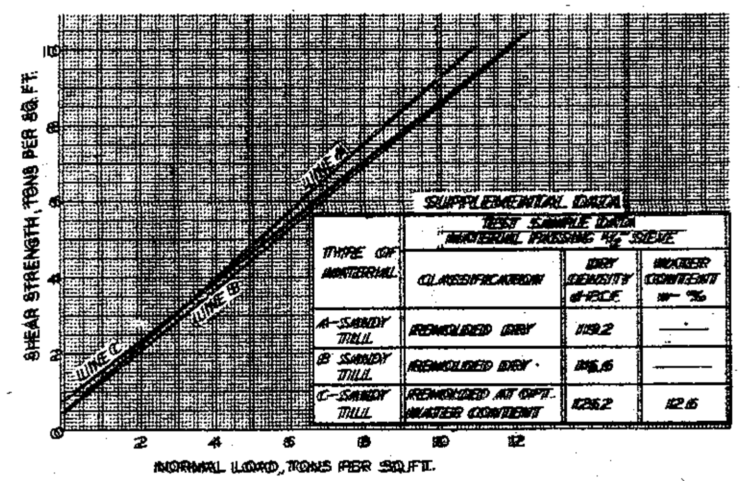
DENSITY RATIO PLOT FOR GRAVELLY AND SILTY SAND FOR RANDOM FILL
FIG. 3



DENSITY RATIO PLOT FOR GRAVELLY SAND FOR PERVIOUS FILL
FIG. 4



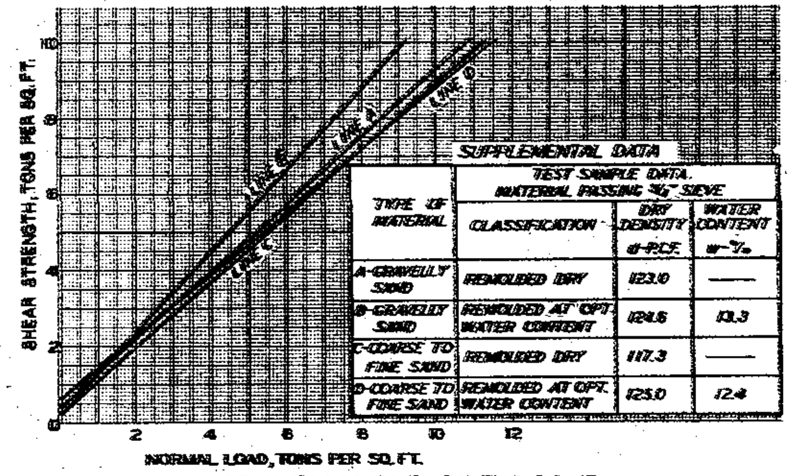
SILTY TILL
(IMPERVIOUS)
FIG. 5



SUPPLEMENTAL DATA

TYPE OF MATERIAL	TEST SAMPLE DATA "MATERIAL PASSING ¾" SIEVE" CLASSIFICATION	DRY DENSITY d-P.C.F.	WATER CONTENT w-%
A-SANDY TILL	REMOLDED DRY	119.2	—
B SANDY TILL	REMOLDED DRY	116.6	—
C-SANDY TILL	REMOLDED AT OPT. WATER CONTENT	126.2	12.6

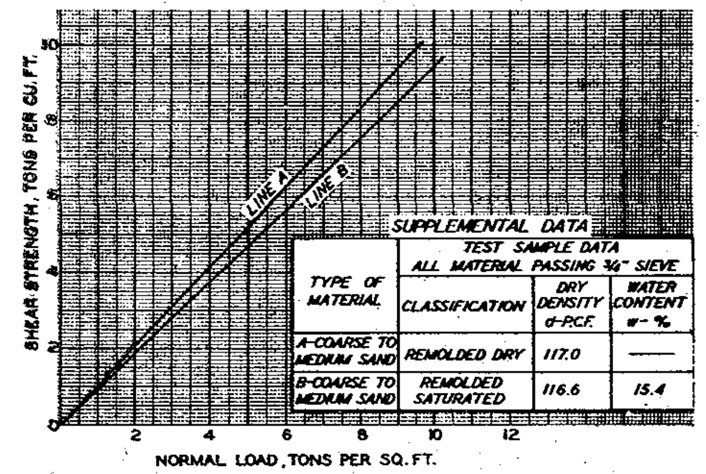
SANDY TILL
(SEMI-IMPERVIOUS)
FIG. 6



SUPPLEMENTAL DATA

TYPE OF MATERIAL	TEST SAMPLE DATA. MATERIAL PASSING ¾" SIEVE CLASSIFICATION	DRY DENSITY d-P.C.F.	WATER CONTENT w-%
A-GRAVELLY SAND	REMOLDED DRY	123.0	—
B-GRAVELLY SAND	REMOLDED AT OPT. WATER CONTENT	124.6	10.3
C-COARSE TO FINE SAND	REMOLDED DRY	117.3	—
D-COARSE TO FINE SAND	REMOLDED AT OPT. WATER CONTENT	125.0	12.4

GRAVELLY AND SILTY SAND
(SEMI-PERVIOUS)
FIG. 7

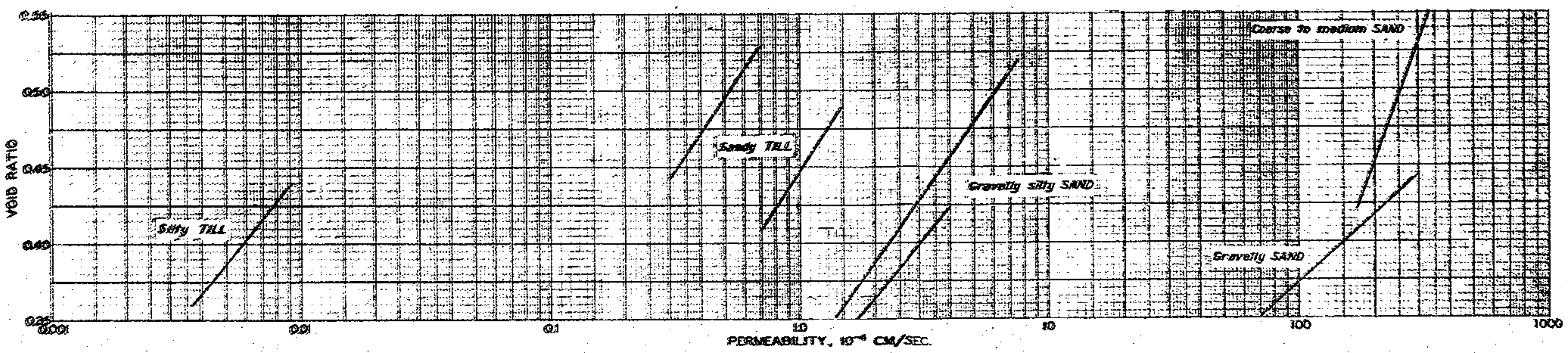


SUPPLEMENTAL DATA

TYPE OF MATERIAL	TEST SAMPLE DATA ALL MATERIAL PASSING ¾" SIEVE CLASSIFICATION	DRY DENSITY d-P.C.F.	WATER CONTENT w-%
A-COARSE TO MEDIUM SAND	REMOLDED DRY	117.0	—
B-COARSE TO MEDIUM SAND	REMOLDED SATURATED	116.6	15.4

GRAVELLY SAND
(PERVIOUS)
FIG. 8

ENVELOPES OF MOHR CIRCLES FOR REPRESENTATIVE SAMPLES OF MATERIAL IN CONSOLIDATED QUICK SHEAR TESTS



PERMEABILITY-VOID RATIO PLOT FOR REPRESENTATIVE SAMPLES OF PRINCIPAL FOUNDATION AND EMBANKMENT MATERIALS
FIG. 9

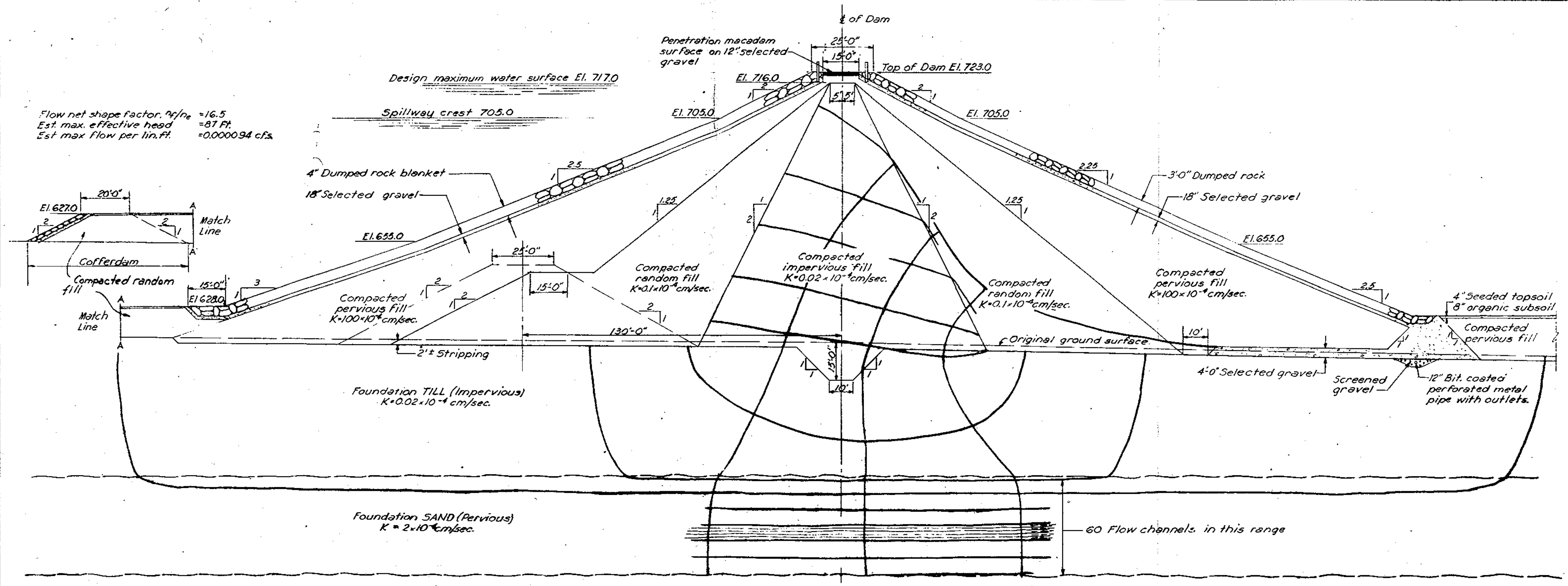
NOTES

All compaction test results, except dry maximum vibrated densities, are based on material passing the No. 4 sieve.
Vibrated densities are determined using material passing the 2 inch sieve.
All shear and permeability test results are based on tests run on material passing the ¾ inch sieve.

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR

SOIL DATA SUMMARY NO. 2

FILE NO. M16-12/60
U.S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945

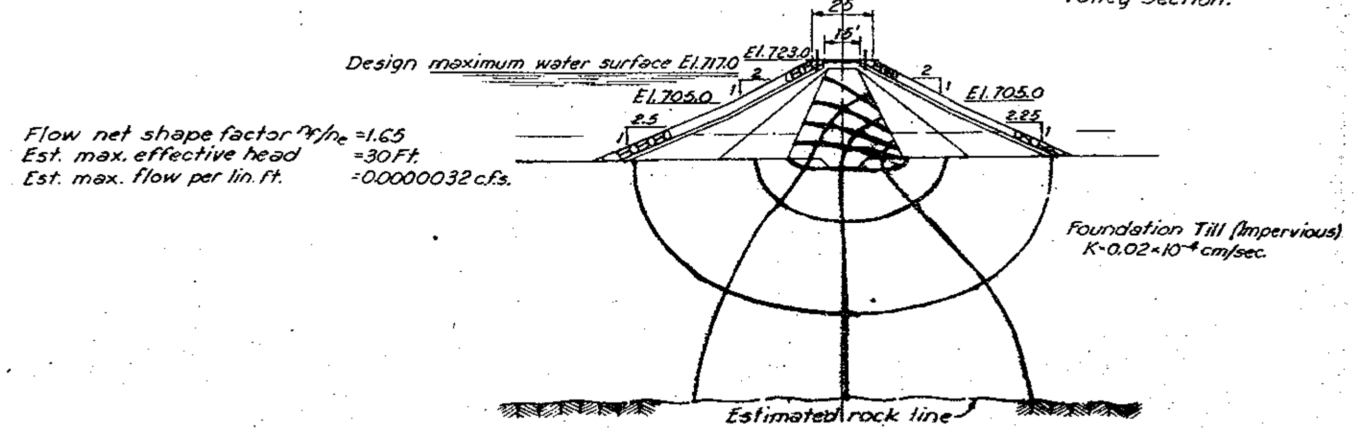


TYPICAL VALLEY SECTION

20 0 20 40 SCALE IN FEET

FIG. 1

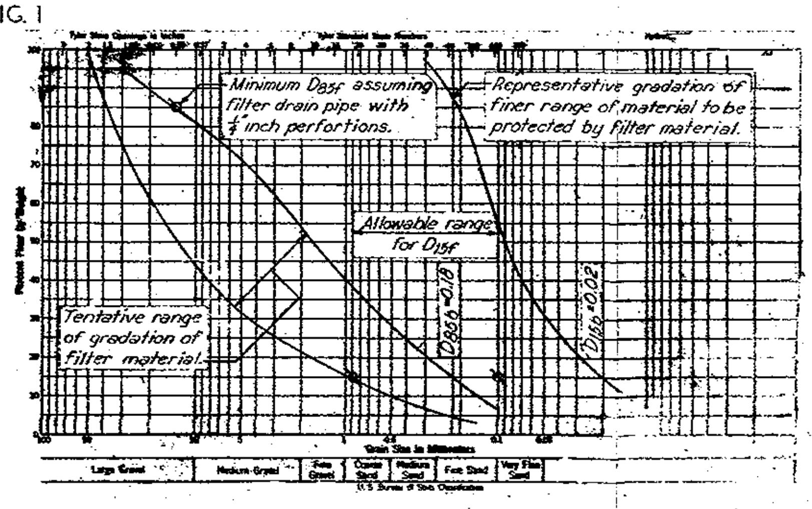
Note:
Embankment materials and K values same as for Typical Valley Section.



CENTERLINE SECTION AT STATION 10+50

40 0 40 80 SCALE IN FEET

FIG. 2

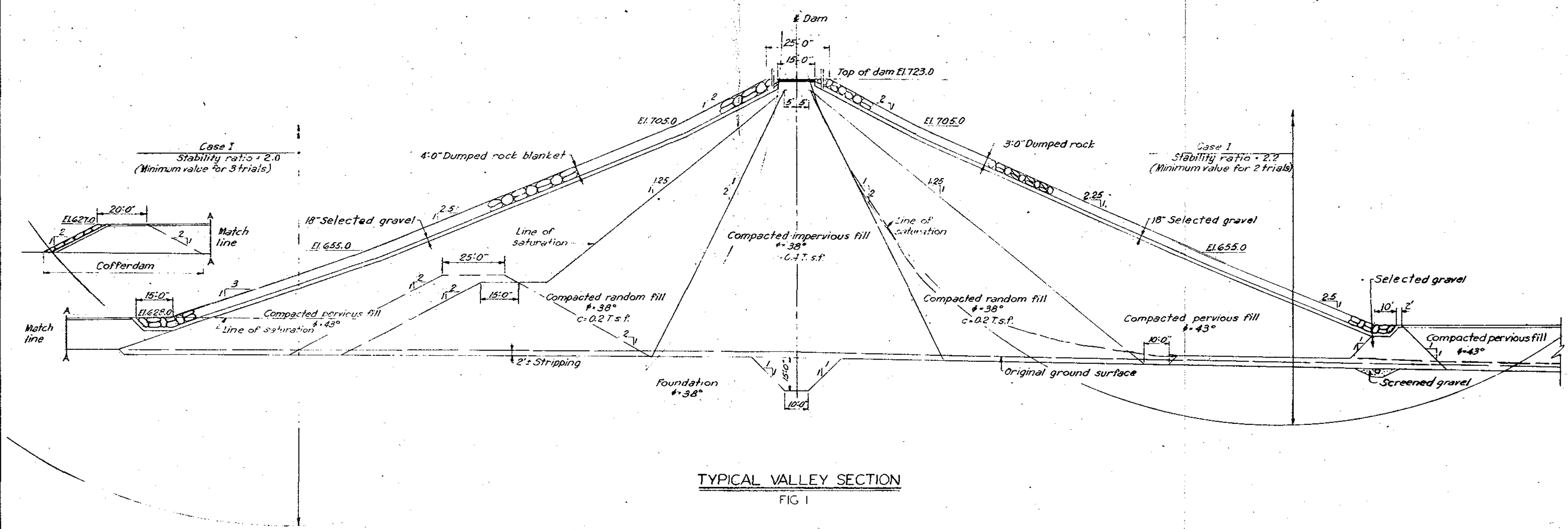


FILTER ANALYSIS

FIG. 3

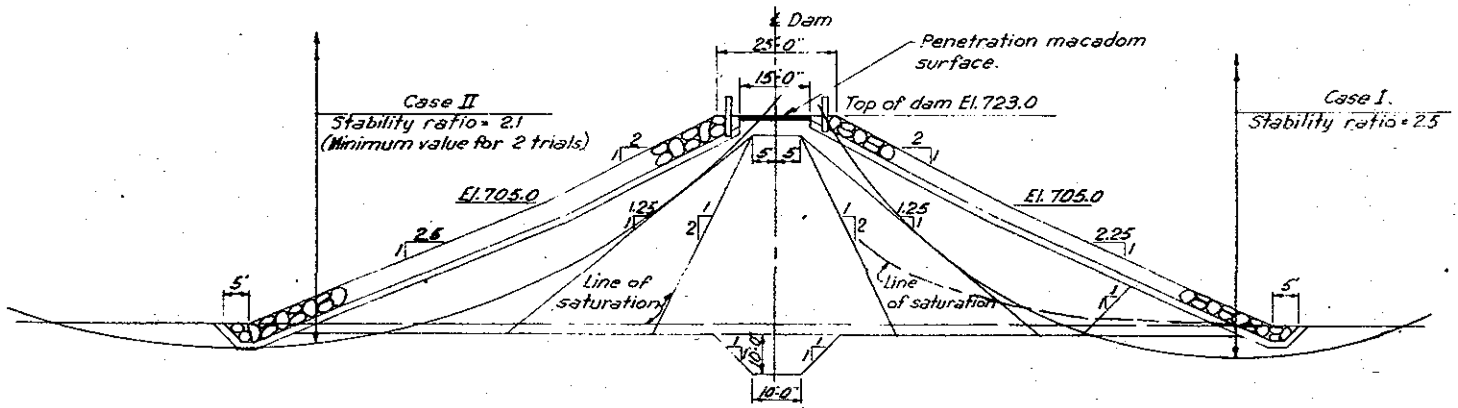
NOTES
a. Impervious core considered stratified with $K_H = 4K_V$.
b. Foundation till considered unstratified.
c. Permeability of foundation sand more than 100 times that of foundation till. For flow net assume $K=2\times10^{-4}$ cm/sec.
d. For flow net study, upstream water surface assumed at El. 710.0 for sufficient time to obtain state of steady flow.

MERRIMACK VALLEY FLOOD CONTROL BEARDS BROOK RESERVOIR
SEEPAGE AND FILTER ANALYSES
SCALE AS SHOWN
U.S. ENGINEER OFFICE, BOSTON, MASS. OCT 1945 FILE NO. [illegible]



TYPICAL VALLEY SECTION

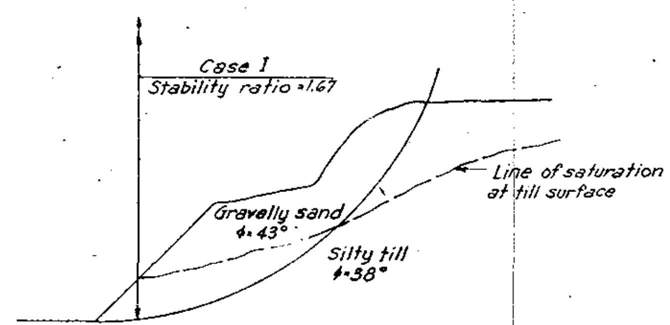
FIG 1



Note:- Embankment materials and shear data same as for Typical Valley Section

TYPICAL SECTION ABOVE EL. 680.0

FIG 2



CONDUIT CONSTRUCTION SIDE SLOPE

FIG 3

NOTES

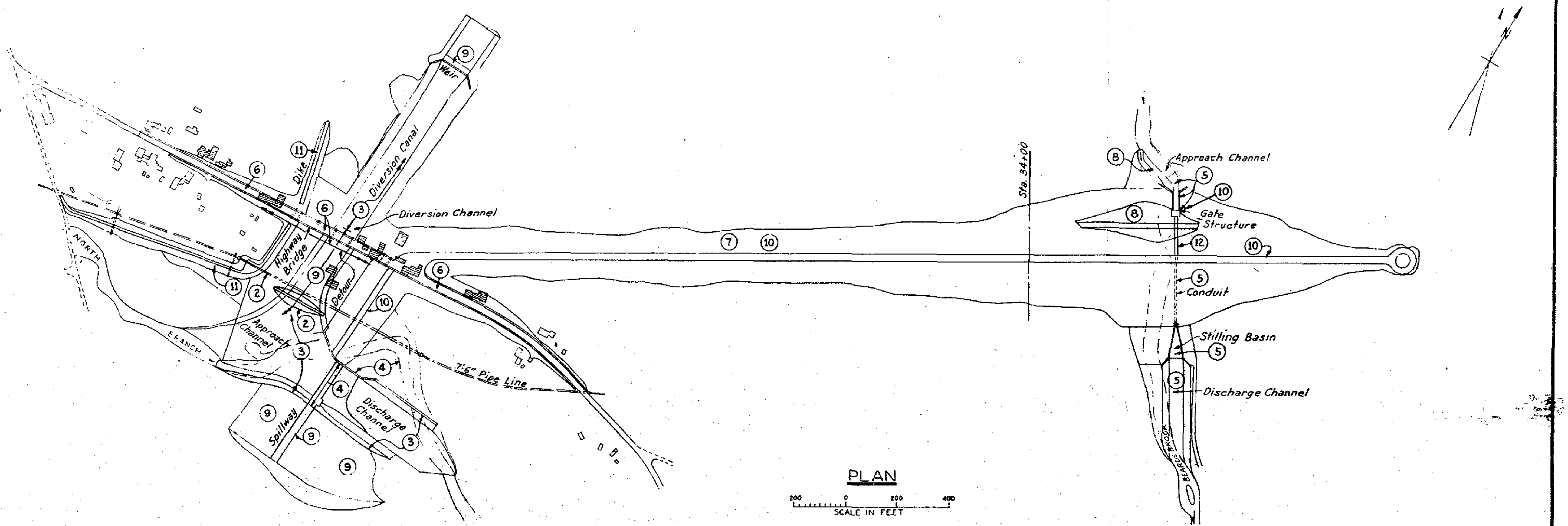
Seepage forces have been neglected in these analyses.
Cases studied are for sudden draw down
Analysis conditions for embankment:-
(a) Upstream slope analyzed for case of sudden draw down of pool from El. 717 to El. 628 with line of saturation as indicated.
(b) Downstream slope analyzed for case of sudden draw down of pool from El. 618 to El. 610 with line of saturation as indicated.

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR

STABILITY ANALYSES

20 0 20 40
SCALE IN FEET

U.S. ENGINEER OFFICE BOSTON, MASS.
OCT. 1945 FILE NO. M16-12/62



SCHEDULE OF CONSTRUCTION OPERATIONS

FIRST SEASON

	DESCRIPTION OF OPERATIONS	APPROXIMATE QUANTITY CU. YARDS
1	Remove existing buildings ---- , Clear and strip site.	Clearing....60 Acres Stripping...145,000
2	Construct 7'-6" diameter pipe line and adjacent cofferdam.	Excavation.... 7,500 Fill .... 5,000 Concrete..... 2,600
3	Excavate diversion canal and portion of diversion channel. Construct portion of spillway approach and discharge channels.	Excavation... 98,000 Fill 13,000
4	Construct portion of spillway to El. 695, entire retaining wall and filled area north of spillway discharge channel.	Excavation.... 2,500 Fill .... 40,000 Concrete..... 5,500
5	Construct conduit, stilling basin and walls; approach channel walls, intake structure to El. 650; portion of approach and discharge channels.	Excavation.... 23,000 Fill .... 30,000 Concrete..... 6,400
6	Construct highway bridge and approaches; detour, using portion of existing state highway	Excavation..... 1,000 Fill .... 28,000 Concrete..... 2,600
7	Construct embankment to El. 705 from Sta. 8+30 to Sta. 34+00.	Excavation... 21,500 Fill 346,000

SECOND SEASON

	DESCRIPTION OF OPERATIONS	APPROXIMATE QUANTITY CU. YARDS
8	Divert Beards Brook thru conduit, construct cofferdams across Beards Brook.	Excavation.... 2,000 Fill 27,000
9	Complete the spillway, spillway approach and discharge channels, diversion channel and diversion canal.	Excavation.. 155,000 Fill .... 50,000 Concrete.... 4,000
10	Complete the intake structure and embankment.	Excavation..... 25,500 Fill .... 844,000 Concrete....... 700
11	Construct dike along diversion channel	Fill 16,000

THIRD SEASON

	DESCRIPTION OF OPERATIONS	APPROXIMATE QUANTITY CU. YARDS
12	Construct access bridge to gate house. Final grading; place subsoil and topsoil. Surface for road on embankment and road to stilling basin.	Lump Sum Fill 15,000 9,000 Sq. Yds.

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

SCALE AS SHOWN

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR

SCHEDULE OF CONSTRUCTION OPERATIONS

U. S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945.

APPROVAL RECOMMENDED: John E. Allen
CHIEF ENGINEERING DIVISION

APPROVED:
COLONEL, CORPS OF ENGINEERS
DISTRICT ENGINEER

SUBMITTED:
CHIEF DESIGN BRANCH, ENGINEER

DES. BY
DR. BY
TR. BY
CH. BY

FILE NO. M16-12/79

APPENDIX III

HYDRAULIC DESIGN

To accompany definite project report
dated November 1945

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX III – HYDRAULIC DESIGN

CONTENTS

PLATES

DEFINITE PROJECT REPORT
BEARDS BROOK RESERVOIR

APPENDIX III

HYDRAULIC DESIGN

a. Introduction.- This appendix presents analyses of the hydraulic design and criteria established for the spillway and outlet structures of the reservoir for the flow conditions varying from normal discharge to the design floods. The hydraulic design is complicated by the fact that the dam extends across two rivers just upstream from their confluence. The spillway is located on the North Branch of the Contoocook River and the principal outlet structure is located on Beards Brook. The general criteria for the hydraulic design of the outlet structures and spillway are as follows:

(1) Normal Flow.- The normal flow in Beards Brook will pass through the Beards Brook conduit without any gate operation. Water from Jackman Reservoir on the North Branch, in excess of the flow utilized at the Jackman Generating Station, will discharge through the ungated outlet in the spillway section into the North Branch.

(2) Freshets.- Freshet flows will discharge similar to the normal flows, that is, each tributary discharge will be maintained in its individual stream without diversion from one to the other. Some storage will probably be used on both tributaries, the amount being dependent on the size of the freshet.

(3) Reservoir Design Flood.- The storage capacity in the reservoir is practically all contained in the section of the reservoir on Beards Brook as the storage in the small pond created on the North Branch is negligible. Consequently during flood periods it is necessary to divert the flood flows from the North Branch to Beards Brook. The ungated outlet in the spillway will limit the discharge from the North Branch, and the diversion canal will have sufficient capacity to divert the excess flow without any spillway discharge. The outlets on Beards Brook will control the discharge during the flood to the prescribed regulated discharge, but still have available discharge capacity to facilitate rapid emptying of the reservoir after the flood.

(4) Spillway Design Flood.- The inflows from Beards Brook during spillway floods are diverted from the Beards Brook section of the reservoir to the spillway on the North Branch. This direction of flow is directly opposite to the

flow in the diversion canal during the reservoir design floods. The diversion channel should have sufficient capacity to pass the spillway flood without causing appreciable head losses.

The design of the hydraulic features of the structures is based on the use of standard formulae and the results of studies and model tests made for similar structures on other projects. The shape of the spillway nappe and the curve for the upstream face were drawn from the recommendation contained in Circular Letter No. 3281, dated 2 September 1944.

The structures have been designed to provide for the discharge of the maximum possible flood stages with the minimum prescribed freeboard and to permit passing normal and freshet discharges with a minimum of gate operation. Three gates are provided in the intake structure to obtain symmetrical flow conditions in the transition and conduit. An eight (8) foot square conduit, and an ungated outlet 6' x 4', in the spillway section are provided for regulation and discharge of all flows below elevation 705 from Beards Brook and the North Branch, respectively. A conduit stilling basin is necessary due to the high discharge velocity and the absence of bed-rock at the conduit outlet. The stilling basin has a depth of 10.0 feet below the end sill; has 2 rows of stepped baffles to dissipate the conduit discharge energy; and has a stepped end sill to provide an adequate tailwater depth for all conduit discharges. The spillway length has been selected as 450 feet for the required discharge capacity due to economical limitations. A diversion canal 1100 feet long is provided to enable storage water from the North Branch to discharge into the Reservoir. The canal, with a capacity of 5000 c.f.s., has a bottom width of 90 feet, side slopes of 1 on 2 and an average depth of 9 feet. A control weir with crest elevation 690 extends across the canal to control the discharge velocity. The diversion channel is trapezoidal in section, 300 feet wide at the bottom and 9.5 feet in average depth, and has a required capacity of 51,500 c.f.s. to pass the spillway flood flows from the reservoir to the spillway section.

b. North Branch Ungated Outlet.- (1) Hydraulic Requirements.- The criteria considered for the design of the ungated outlet on the North Branch are summarized as follows:

(a) To discharge normal flows in the North Branch without diversion to Beards Brook. It is desired to eliminate diversion, except during flood conditions, as there is no well-defined stream bed connecting the two rivers.

Continuous diversion without any storage pool in the Beards Brook section of the reservoir would result in erosion and silting problems in Beards Brook.

(b) To restrict the discharge on the North Branch in order to control flood flows.

(c) To facilitate construction of the spillway and to avoid diversion of water during construction.

(2) **Selection of Size.**- A 6' x 4' outlet through the concrete spillway section was selected to satisfy these conditions. The discharge capacity of the outlet with pool at elevation 690 is approximately 500 c.f.s. This is the pool elevation that can be maintained on the North Branch portion of the reservoir without diversion to Beards Brook. As may be noted on the hydrograph of the North Branch (Plates I-4 and I-5), flows exceeding 500 c.f.s. are infrequent and are approximately of a one year frequency. The discharge capacity of the outlet with pool elevation at the spillway crest is about 860 c.f.s.

(3) Outlet Discharge Curve.- The discharge curve for the North Branch outlet is shown on Plate III-1. The lower part of the discharge curve was obtained by assuming channel flow in the outlet with critical flow conditions at the entrance. The upper portion of the curve was computed by summating the losses for the entrance, friction, and velocity head. The sum of these losses result in a coefficient of discharge equal to 0.93 in the orifice and tube formula, $Q = CA\sqrt{2gH}$

where Q = discharge in c.f.s.
C = coefficient of discharge
A = cross-sectional area in square feet
g = accellaration due to gravity
H = head on outlet in feet (measured to top of portal)

(4) Elimination of Gates.- Gates will be omitted on this outlet to eliminate the necessity for a gate control structure and to simplify the proposed method of gate operation during flood conditions. All discharge regulation, to augment the flow from the ungated outlet, will be controlled with the gates provided in the conduit on Beards Brook.

c. Beards Brook Conduit.- (1) Hydraulic Requirements.- The Beards Brook conduit and gate capacities are designed to meet the following criteria:

(a) To discharge approximately 1800 c.f.s. with reservoir at spillway crest, which combined with the discharge from the North Branch ungated outlet is approximately equal to the safe downstream capacity.

(b) To discharge approximately 2500 c.f.s. with the reservoir at elevation 690 to expedite emptying the reservoir following the flood. It should be noted that between elevations 690 (crest of weir in diversion canal) and 705 the reservoir will discharge through the ungated outlet on the North Branch in addition to the discharge from the Beards Brook conduit. Below elevation 690, the reservoir must be entirely emptied through the Beards Brook conduit.

(c) To discharge approximately 2500 c.f.s. with reservoir stage at elevation 665. This criteria is desirable to allow a discharge from the reservoir equivalent to the downstream channel capacity utilizing only 25% of the total reservoir storage.

(d) To discharge approximately 1500 c.f.s. with reservoir stage at elevation 645 to provide capacity at low heads in case the reservoir should ever be used for conservation purposes.

(e) To obtain discharge criteria at the higher reservoir stages without resorting to partial gate opening.

(2) Selection of Size.- To meet these criteria a square conduit 8 feet wide and 8 feet high was selected. This cross-sectional area is necessary to satisfy condition (c) above which is the governing condition for the conduit capacity. The portal of the conduit is rectangular in cross-section with a width of 9'-6", and a height of 6'-6". This results in an area reduction from 64 to 61.75 square feet and assures positive pressures within the conduit. Reducing the height of the conduit and flaring also tends to initiate a spread in the discharge jet that continues into the stilling basin. Three gates are provided; the center gate 4 feet wide and 7 feet high, and the outside gates 3 feet wide and 7 feet high. The 7 x 4 gate has sufficient capacity to satisfy condition (a) and the two 7 x 3 gates will satisfy condition (b). The selection of gate sizes is influenced in part by the ungated outlet on the North Branch in order to keep the flow below the confluence of the North Branch and Beards Brook within the channel capacity. It is also to be noted that this North Branch outlet is only effective in emptying the reservoir when the pool is above elevation 690. The conduit has a total length of about 440 feet of which approximately 20 feet will be used in the gate passages. The invert grade of the

conduit which will slope from elevation 617 at the entrance to elevation 613 at the portal, is selected primarily to meet foundation conditions.

(3) Conduit Discharge Curves.- The relation of reservoir elevation to conduit discharge is shown on Plate III-2. This is a compound curve with the upper portion computed for the conduit flowing full with control at the portal. The lower portion is computed for the conduit flowing partly full. The uncertain zone where these curves intersect has been estimated to agree with discharge curves obtained from model studies of comparable conduits. For the conduit flowing full, the hydraulic losses were computed in terms of exit velocity head. Friction losses were computed by the Manning formula using n = 0.013. The summary of losses in terms of portal velocity head for discharge with all gates open are as follows:

Entrance	0.06	$\frac{Vx^2}{2g}$
Gate passages, gate slots, transition, etc.	0.12	"
Friction in conduit	0.88	"
Velocity head at portal	1.00	"
Total head "H"	2.06	"

The discharge can be expressed in terms of the head by the follow- equation:

$$Q = 364 \sqrt{H}$$

where Q = discharge in c.f.s.

H = head in feet as measured from the reservoir stage to the roof of the portal.

(4) Discharge Capacity of Gates.- The discharge capacity with two gates open is computed similar to the method outlined for all gates open. With one gate open the single gate passage will be the hydraulic control instead of the conduit portal. The discharge is computed similar to the method outlined for all gates open. The discharge is computed considering entrance, gate, friction, and velocity head losses in terms of the velocity of flow in the gate passageway. The lower portion of the rating curves is computed by assuming critical flow conditions at the entrance to the gate passages. The energey and hydraulic gradients in the conduit with all gates open and the reservoir at spillway crest elevation are shown on Plate III-3. Cross sections of conduit and portal are also shown.

(5) Discussion of Gate Sizes.- An attempt has been made to select gates of the same size to facilitate construction, installation, and inter-change of parts, but it was found that three similar gates did not have the flexibility of discharge regulation to satisfy the design criteria. The high head on the gates (head equals 88 feet with water at spillway crest) makes it undesirable to operate gates at partial openings because of the possibility of vibration and cavitation from the negative pressures. Three gates of equal size (say, 7 x 3-1/2 feet) result in a discharge capacity for one gate that is too small for controlling the reservoir design flood (Flood of March 1936), as described in Appendix I, without appreciable spillway discharge. Two gates open for low reservoir stages result in too high a rate of discharge for the frequent floods and not optimum utilization of the reservoir storage for the maximum downstream flood reductions. The selected gate sizes provide a wide range in rates of discharge without resorting to partial gate openings as indicated on Plate III-2. This range in available discharge is advantageous in emptying the reservoir following a flood as shown on Plate I-26. The discharge equal to the downstream channel capacity can be maintained more uniformly and consequently decrease the time necessary to empty the reservoir. Routing computations indicate that the time required to empty the reservoir using three equal gates without partial opening is approximately 1-1/2 days longer than the time taken by the selected gates. Even more important is the fact that with the proposed gates an additional inch of storage is made available at the end of the 7th day by being able to closely maintain the channel capacity. The discharge range provided by the selected gates is also believed desirable in case the reservoir is utilized for conservation purposes in the future when it may be necessary to discharge prescribed rates for downstream use. It is estimated that the additional cost for providing two sizes of gates is approximately $3000, but it is considered that the merits and advantages of the selected gate sizes justify this additional expenditure.

d. Conduit Stilling Basin.- (1) General.- The absence of bed-rock in the vicinity of the conduit outlet and the high velocity of discharge that will result with any appreciable reservoir stage make it necessary to provide a stilling basin to dissipate the energy in the conduit discharge. The reach of river below the outlet consists of a well defined channel lined with boulders and with fairly steep side slopes. The river flow is turbulent due to the effect of the boulders and the grade which averages approximately 1 percent. There is no natural channel restriction or hydraulic control to provide a stabilized tailwater.

(2) Hydraulic Requirements.- The stilling basin is designed to insure the formation of an hydraulic jump under the following discharge conditions:

(a) Reservoir stage - Elevation 718
(b) Conduit discharge
(Based on "n" = 0.011) = 3860 c.f.s.
(c) Portal velocity = 61 ft.per sec.

(3) Design of Stilling Basin.- The stilling basin follows conventional patterns of sloping and stepped floor and expanding sections. The curve for the floor of the expanding section from the portal to the floor of the stilling basin is determined by the trajectory of the jet of water leaving the exit portal. To insure flattening of the jet and complete filling of the expanding section, the initial horizontal velocity of the jet is taken as 70 feet per second, or about 9 percent greater than the maximum computed velocity. The equation of this parabolic trajectory is $X^2 = 800y$. Since the jet falls 13 feet (elevation 613 to 600), the required length of the expanding section is 72.5 feet. The depth of tailwater required for obtaining an hydraulic jump is based on the formula:

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{2v_1^2 d_1}{g} + \frac{d_1^2}{4}}$$

where d_2 = depth of flow in feet after the jump
d_1 = depth of flow before the jump
v_1 = velocity of flow in feet per second before the jump
g = acceleration due to gravity.

The required depth, for a stilling basin width of 40 feet, is approximately 18.5 feet. The expanding section has a gradual flare that increases the width from 9 feet 6 inches at the conduit exit portal to 40 feet at the upstream end of the stilling basin. The width continues to expand to a maximum width of 60 feet in order to obtain a lower depth of flow over the end sill and in the discharge channel. The stilling basin has two rows of baffles and a high stepped end sill at elevation 610. The plan and profile of the stilling basin with a discharge of 3860 c.f.s. are shown on Plate III-4.

(4) Tailwater.- The stilling basin end sill is designed to establish a definite tailwater rating curve as there is no natural control to cause a determinable stage-discharge

relationship. The end sill corresponds to a flat-crested weir with an assumed maximum coefficient of discharge equal to 3.2. The discharge channel is designed with sufficient cross sectional area and bottom slope to insure that the hydraulic control remains at the end sill. The end sill will provide an adequate tailwater depth for all variations in conduit discharge as shown on Plate III-4.

e. Spillway.- (1) General.- The concrete spillway located on the North Branch will provide discharge capacity for the spillway design floods originating on both the North Branch and Beards Brook. Furthermore, the spillway will have additional capacity for the discharge that would come from the possible failure of Jackman Dam during the super-floods.

(2) Design.- The spillway length of 450 feet is selected as the most economical length to provide the required discharge capacity. The length of the embankment section precludes shortening the spillway and increasing the height of the dam, for the cost of the embankment section rises rapidly per foot of additional height. To satisfy the discharge requirements shown on Plate I-21, the spillway is designed for discharge capacity of 80,000 c.f.s. with a design head of 13.0 feet. The discharge rating curve shown on Plate III-5 is based on the weir formula:

$$Q = CLH^{3/2}$$

where Q = discharge in c.f.s.
C = Coefficient of discharge
L = Length of weir in feet
H = Head on weir in feet

The coefficient "C" is assumed to vary from 3.0 to 3.8 depending on the ratio of the effective head to the design head as shown on Plate III-5. The velocity of approach to the spillway is neglected due to the uncertainties in the flow condition approaching the spillway from the diversion channel and the North Branch.

(3) Spillway Nappe.- The shape of the spillway conforms to the exponential curve outlined in Circular Letter No. 3281, dated 2 September 1944, subject: "Shape of Spillway Crests." The curve is expressed as follows:

$$X^{1.85} = 2 H_c^{0.85} Y$$

where X = horizontal distance from ogee crest line
Y = vertical distance below ogee crest level
H_c = design head on ogee crest

For a design head of 13.0 feet this function reduces to:

$$X^{1.85} = 17.70Y$$

The curve for the upstream face of the dam to the crest is a compound curve conforming to similar recommendations in the above Circular Letter.

(4) Tailwater.- The channel downstream from the spillway will be a sloping rock cut. No stilling basin is required as the energy in spillway discharges will be dissipated in turbulent flow. There may be some soil erosion at the bend in the channel (Plate IV-2) but there can be no damage done to any of the dam structures.

f. Diversion Canal and Channel.- (1) General.- The trapezoidal cut connecting the two sections of the reservoir and utilized primarily for the flow from Beards Brook to the spillway is designated as the Diversion Channel. The smaller trapezoidal cut in the bottom of the diversion channel, and used for diverting flood flows from the North Branch to Beards Brook is designated the Diversion Canal. It should be noted that the flow in the canal and the channel are in opposite directions during their respective design conditions.

(2) Hydraulic Requirements.- The criteria for the size and capacity of the diversion canal and channel are as follows:

(a) Diversion Canal.-

1. To divert 5000 c.f.s. from the North Branch to the reservoir storage which is principally located on the Beards Brook section of the reservoir. Flows in the canal will occur during the normal operation of the reservoir for discharges on the North Branch that exceed the capacity of the ungated outlet in the spillway. It is anticipated that this condition will occur approximately once a year. The capacity of 5000 c.f.s. is selected to satisfy the anticipated maximum discharge on the North Branch during the reservoir design flood. The peak of the March 1936 flood at the North Branch gaging station (Drainage area = 54.8 square miles) was 4680 c.f.s. Increasing in the ratio of drainage areas, the discharge at the dam site would be 5460 c.f.s. Records of the N.H. Public Service Co. indicate that the release of flashboards on the Jackman Dam resulted in an estimated instantaneous discharge of 8300 c.f.s. which gradually receded to equal the natural flow. It is not believed necessary to design the diversion channel for this higher discharge from the failure of flashboards, but to allow overflow in the bottom of the diversion channel during these conditions, which, if they do occur, will be of short duration. It should be noted that, when the diversion canal is conveying 5000 c.f.s., without submergence, the ungated outlet will be discharging 700 c.f.s., hence, the combination provides for a total flow on the North Branch of 5700 c.f.s.

2. To maintain comparatively low velocities in the canal to prevent erosion.

3. To allow water to flow in the opposite direction without restriction during spillway discharges.

(b) Diversion Channel.- 1. To divert 51,500 c.f.s. (Plate I-22) from Beards Brook section of the reservoir to the spillway without creating velocities which would damage the embankment slopes, the highway bridge, or the relocated pipe line to the Jackman Power Plant.

2. To **function** satisfactorily during the various discharge conditions that would arise with the possible failure of the Jackman Dam during the extraordinary floods causing spillway discharge.

(3) Design of Diversion Canal.- (a) Description.- The diversion canal will be a trapezoidal cross-section with a **bottom** width of 90 feet, side slopes of 1 on 2, and an average depth below the bottom of the diversion channel of approximately 9 feet. The length of the canal is approximately 1100 feet with the invert varying from elevation 689 at the power pipe crossing to elevation 686 at the control weir. (See Plate IV-2). A control weir, with crest length of 110 feet will be constructed at the end of the canal to control the velocity of flow in the channel and eliminate securing and erosion, that otherwise would occur. The crest of the weir will be elevation 690 which will also prevent diversion of the North Branch flows to Beards Brook until the ungated outlet in the spillway is discharging under an appreciable head.

(b) Water Surface Profile.- The water surface profile for a discharge of 5000 c.f.s. through the diversion canal is plotted on Plate III-6. The water surface in the North Branch section of the reservoir for this discharge will be elevation 697.5. The water surface profile was computed by standard backwater methods starting with the hydraulic control at the canal weir and assuming a Kutter's coefficient of friction "n" = 0.040. The average velocities in the canal for the discharge of 5000 c.f.s. will vary from about 5 feet at the weir to about 6 feet at the North Branch end of the canal.

(c) Canal Weir.- The canal weir is lengthened from the canal bottom width of 90 feet to 110 feet to provide sufficient discharge capacity for the prescribed 5000 c.f.s. without overflowing the sides of the canal. The shape of the weir conforms to the same criteria outlined in paragraph (e) of this appendix in order to obtain a high coefficient of discharge. It is assumed that tho coefficient is 3.6 for the design head of 5.5 feet. The formula for the exponential function for the spillway nappe is:

$$X^{1.85} = 8.52Y$$

The discharge energy will be dissipated in a dumped rock area before flowing overland to the reservoir pool. To aid in the dissipation, the flow over the spillway nappe will discharge on to a horizontal concrete toe with concrete baffles to cause further turbulence. It is not expected to form a hydraulic jump but to break down the energy by turbulence. If the flood is of sufficient magnitude to fill the pool in the Beards Brook section above elevation 690, the weir will become submerged, but under these conditions the weir is unnecessary, for the velocities in the canal, or even overflow into the channel, will be adequately controlled and reduced by natural backwater.

(4) Design of Diversion Channel.- (a) Description.- The diversion channel will be a trapezoidal cross-section with a minimum bottom width of 300 feet with side slope of 1 one 3 on the easterly embankment, and 1 on 2 on the westerly dike. The bottom of the channel varies between elevations 698 and 700, which averages 6 feet below the crest of the spillway. The diversion canal extends through the bottom of the channel as shown on Plate IV-2. The length of the channel with the 300 foot width is approximately 400 feet. Beyond this section the width increases to meet the general topography.

(b) Water Surface Profiles.- Plate III-6 shows the profiles of water surface and energy gradients for the channel flow conditions illustrated on Plates I-22 and I-23. The hydraulic assumptions and pertinent data relative to these profiles are summarized as follows:

1. Profile "A" is based on the channel flow conditions illustrated on Plate I-22 and is the maximum flow in the channel from Beards Brook to the North Branch occurring with the assumed failure of the Jackman Dam during the spillway design flood. This condition results in the highest water surface elevations in both sections of the reservoir and leads to the determination of the top of the dam. The grade of the energy gradient in the North Branch section will be 717.9 with a corresponding water surface at elevation 718.9 in the Beards Brook section. The elevation of the energy

gradient is quoted for the North Branch section because of the uncertainties in the true water surface elevation.

2. Profile "B" is based on the channel flow conditions illustrated on Plate I-22 and is the maximum flow in the channel from Beards Brook to the North Brach occurring without failure of the Jackman Dam during the spillway design flood. This condition results in the maximum channel velocities for this particular direction of flow which vary between 5.0 and 7.4 feet per second. The grade of the energy gradient in the North Branch section will be elevation 716.0 with the water surface in the Beards Brook section at elevation 717.2.

3. Profile "C" is based on the channel flow condition illustrated on Plate I-23. It is the maximum flow in the channel from the North Branch to Beards Brook occurring during the computed spillway flood derived from centering the maximum rainfall intensity over the drainage basin of the North Branch. It is assumed that the channel flow of 10,000 c.f.s. occurs without any backwater effect from the Beards Brook section of the reservoir. It is to be noted that the canal will be conveying most of the discharge with only a small depth of overflow on the bottom of the channel, and due to the difference in the botton elevation the depth will not be great enough to cause flow at the toe of the embankment section. The maximum average velocity is computed to be approximately 8 feet per second.

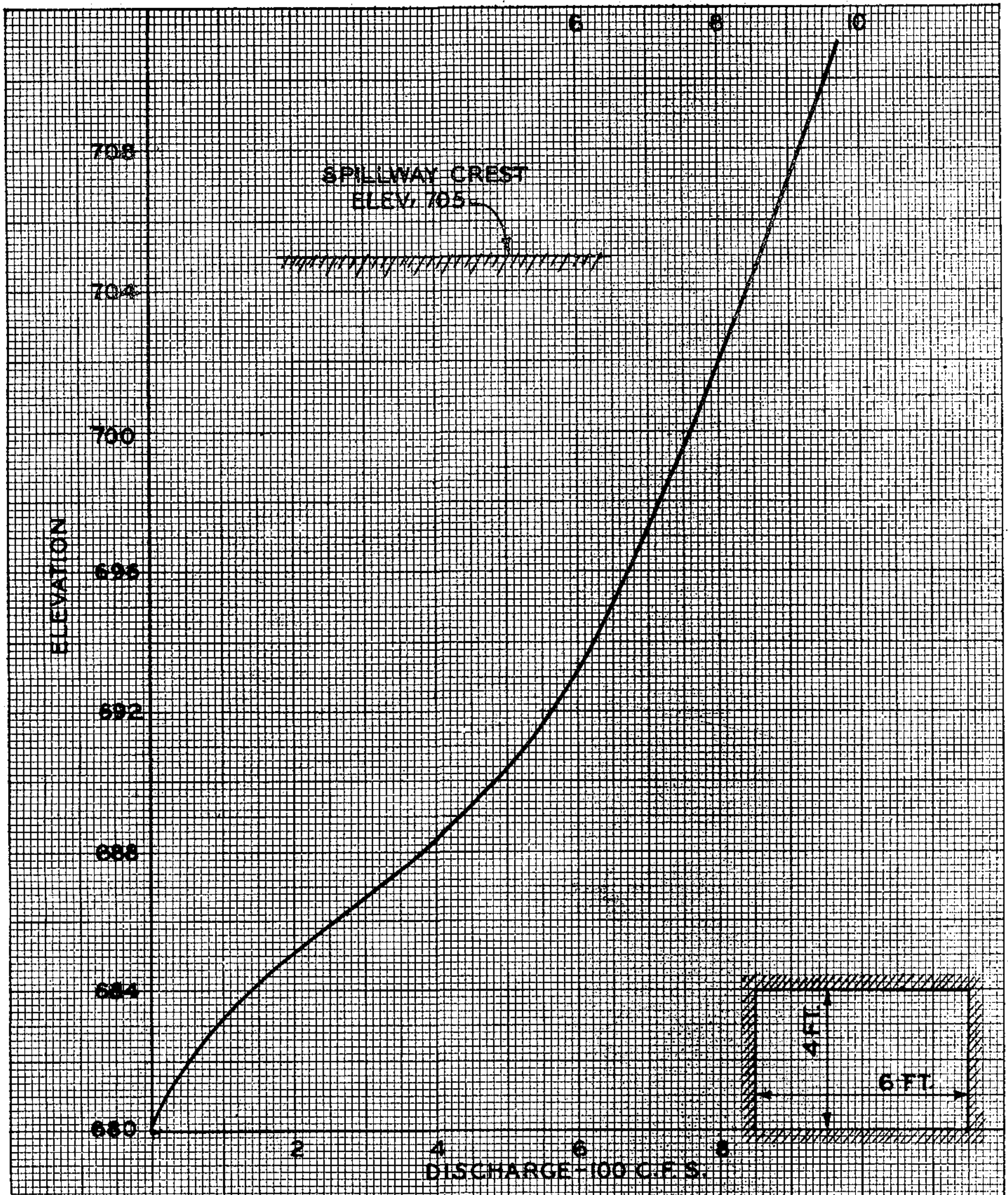
4. Profile "D" is derived from the canal design criteria and is discussed in paragraph f.(3)(b) of this appendix.

(c) Other Design Considerations.- The canal and channel must be adequate to satisfy the many different assumptions and conditions of flow as enumerated in the preceding paragraph. These conditions arise during both the reservoir and spillway design floods. It is possible to conceive a critical condition of flow in the diversion channel resulting from a combination of several improbable circumstances, but it is not possible to mathematically determine the magnitude of the discharge, water surface profiles or velocity distributions that would result. This flood situation would develop similar to the conditions outlined on Plate I-23, namely: the reservoir would be empty at the beginning of a flood-producing storm with rainfall intensities comparable to the limiting rainfall conditions shown on Plate I-14. The greatest intensity of the storm would be assumed to fall on the drainage area of the North Branch in order to produce the severest flood possible on the

North Branch and a relatively smaller flood on the Beards Brook. With this combination of assumptions it is conceivable that the flood on the North Branch would be of such magnitude to cause failure of the Jackman Dam at a time when the storage level in the Beards Brook section of the reservior was below elevation 690 (crest of canal weir). The discharge of water from the failure gap would cause a surge of water flowing down the North Branch. Without depth of storage in the reservoir "to cushion" the rush of water, the surge will be partly diverted through the diversion channel with the remainder discharging over the spillway as the depth would rapidly rise. The valley storage and storage in the North Branch section of the reservoir would tend to reduce the onrush of water, but as both storage factors are small the effect is perhaps negligible. The cross-sectional area of the channel is entirely adequate to pass any conceivable discharge with ample freeboard. The only danger to the embankment section would come from high velocities of flow and the effect of turbulence on erosion and destruction of riprap pavement. It is impossible to hydraulically determine the flow conditions that would develop during these hypothetical circumstances, but it is believed they are of sufficient importance to be analyzed by hydraulic model studies. (See paragraph g.)

g. Model Study.- Due to the undeterminable discharge conditions that will occur in the North Branch section comprising the spillway and its approaches, the diversion canal and channel, and the North Branch, it is believed desirable that a hydraulic model study be made of this section. Information would be obtained on magnitude, direction, and distribution of discharge velocities, adequacy on height of walls and riprap, head losses in the spillway approach and diversion channel, and the determination of the safe height of the embankment section. The model tests should include both the discharge conditions from the selected spillway design flood with and without the assumed failure of the Jackman Dam. It is recommended that the model tests be made at the U. S. Experimental Station at Vicksburg, Miss., to take advantage of their ideal hydraulic model facilities and to profit by the experience and judgment of the personnel at the Station.

h. Weir for Recreation Pool.- Plate VII-1 shows details of a proposed weir that may be used in the design in case the State of New Hampshire advocates a recreation pool. The weir would allow reservoir operation similar to the method discussed in Appendix I, that is, the center 7' x 4' gate will be maintained open, the two 7' x 3' gates will be normally closed, except for emptying purposes following a flood. The control weir will be constructed to the selected elevation of the recreation pool (tentatively determined to be elevation 640) and will be an extension of the two gate piers. Due to space limitations the shape of the weir will not conform to spillway nappe requirements, but it is not considered too essential to have a high coefficient of discharge. It is estimated that the weir will "drown out" with a head of approximately 3.5 feet. Under full head conditions the weir will add slightly to the entrance losses but not to any serious extent. The average downward velocity through the weir opening with the reservoir at elevation 705 is approximately 9 feet per second. The end wall of the weir would be initially constructed to elevation 622 in order to permit full utilization of the gates and conduit capacity during the construction period involving the use of the cofferdam and diversion of the flow through the conduit. The loss of 1000 acre feet of storage for recreation purposes has but slight effect on the reservoir design floods, and would raise the maximum water surface only a few tenths of a foot.



MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

DISCHARGE CURVE
UNGATED OUTLET, NORTH BRANCH

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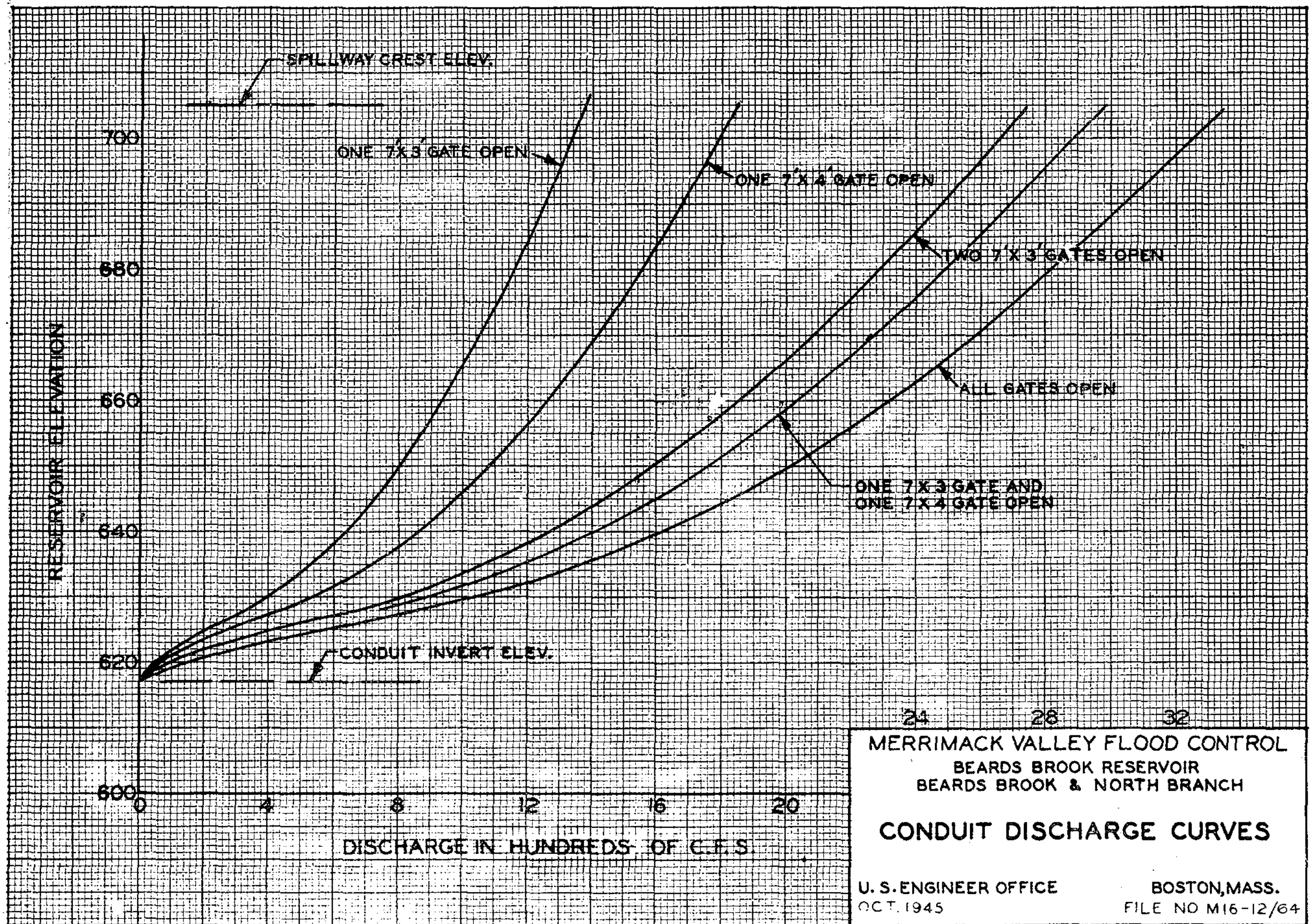
PLATE III - 1



MERRIMACK VALLEY FLOOD CONTROL

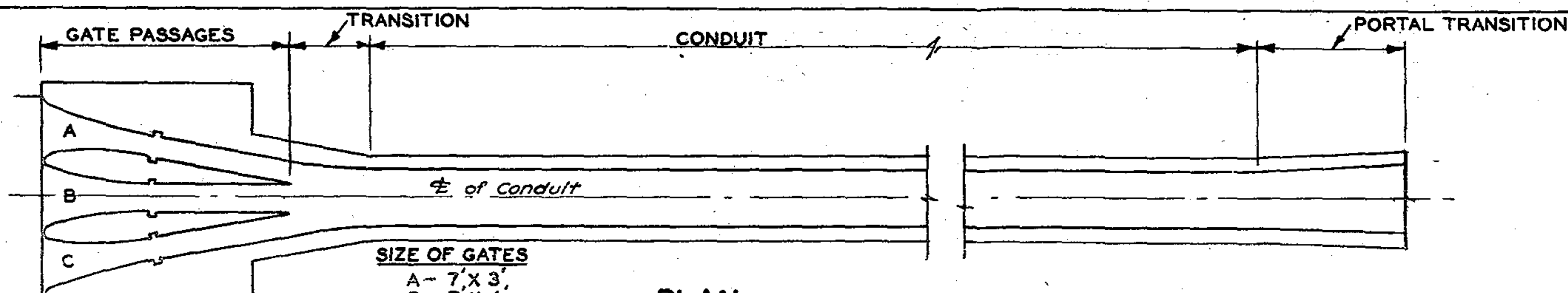
BEARDS BROOK RESERVOIR

BEARDS BROOK & NORTH BRANCH

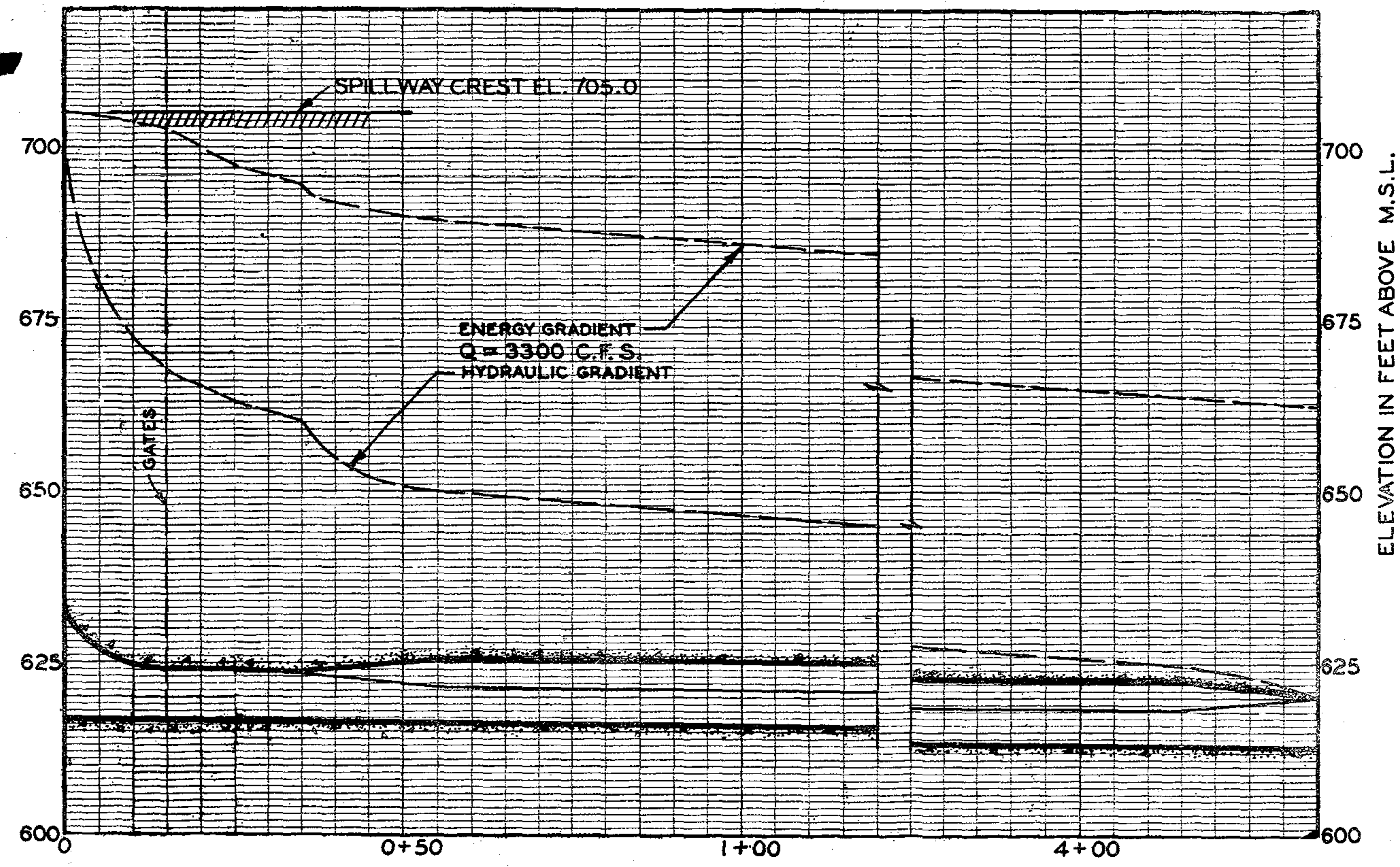
CONDUIT DISCHARGE CURVES

U. S. ENGINEER OFFICE — BOSTON, MASS.

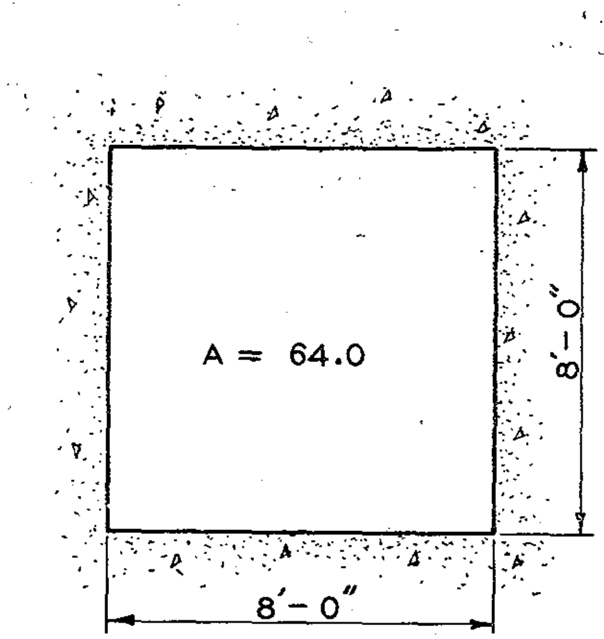
OCT. 1945 — FILE NO M16-12/64



PLAN



STATIONS IN CONDUIT



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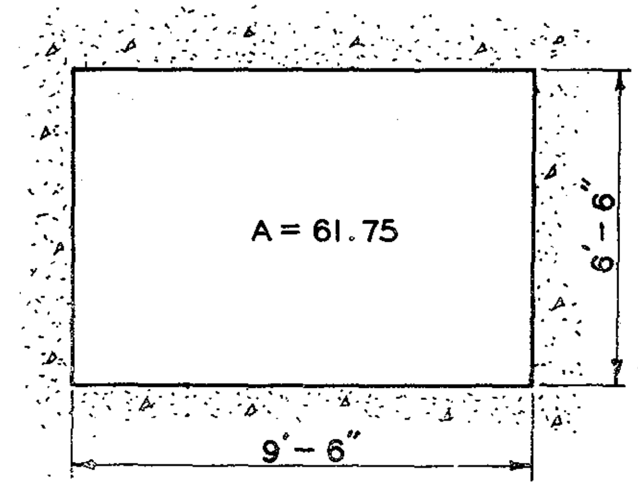

PORTAL

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

CONDUIT GRADIENTS

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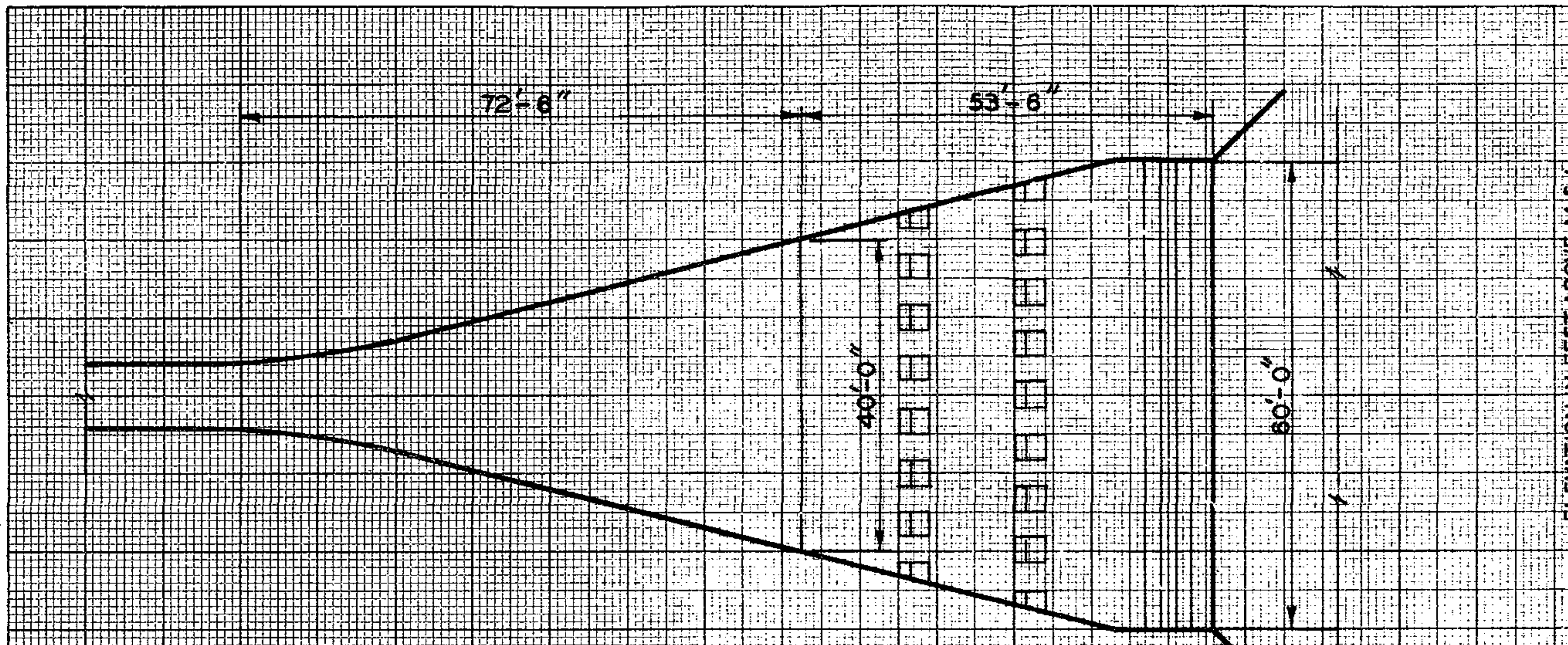
PLATE III - 3



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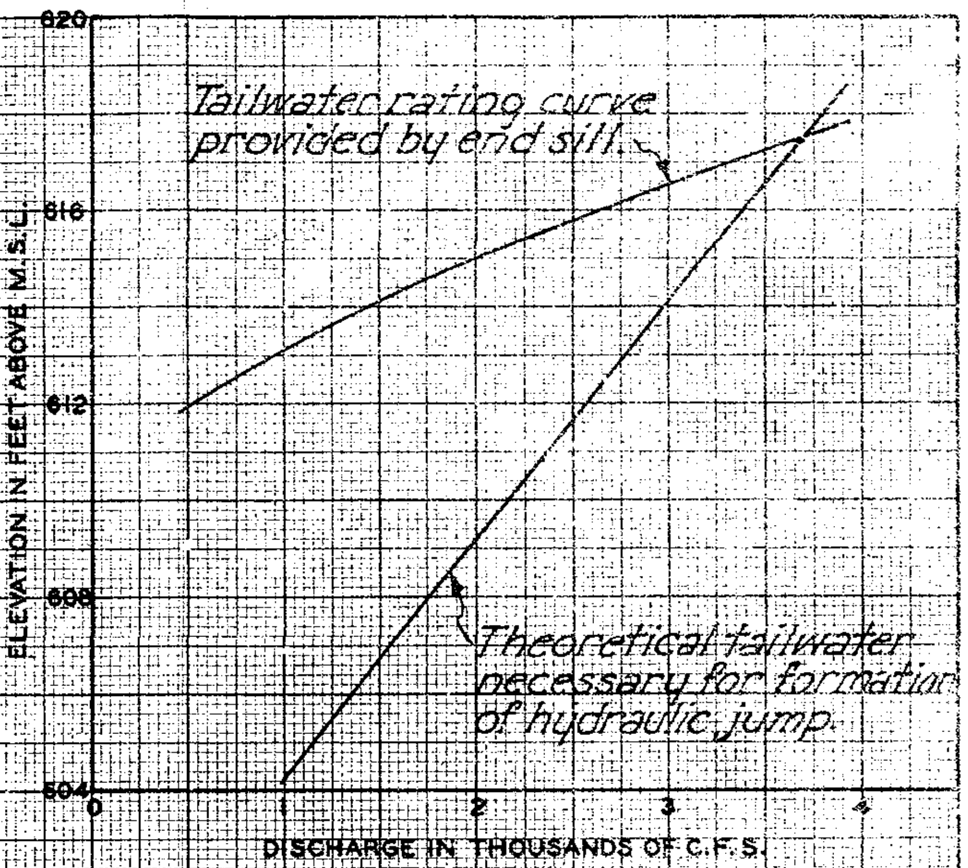

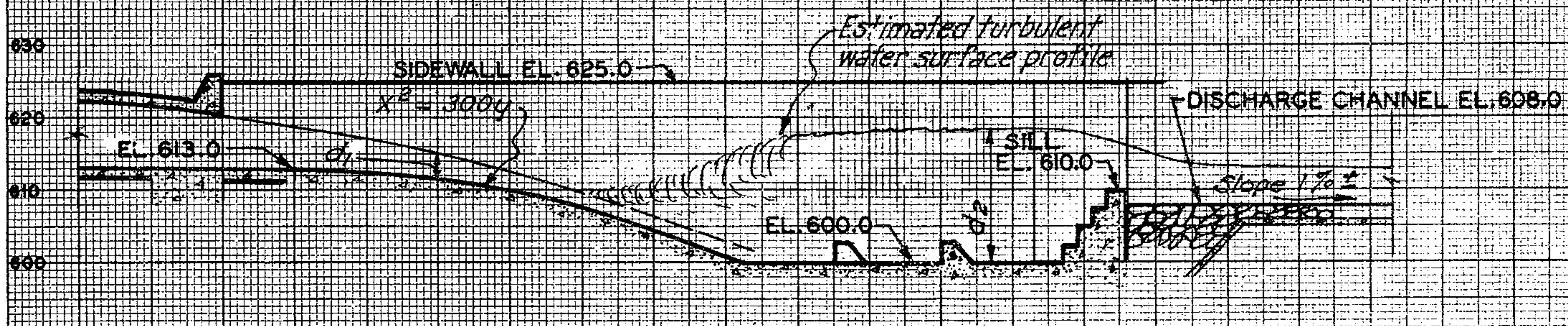

SECTION

SCALE: 1" = 20'-0"

Formula for hydraulic jump:

$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{2v_1^2 d_1}{g} + \frac{d_1^2}{4}}$$

where d_1 = depth above hydraulic jump in feet.
d_2 = " below " " "
v_1 = Velocity above " " "
g = acceleration due to gravity.

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

STILLING BASIN

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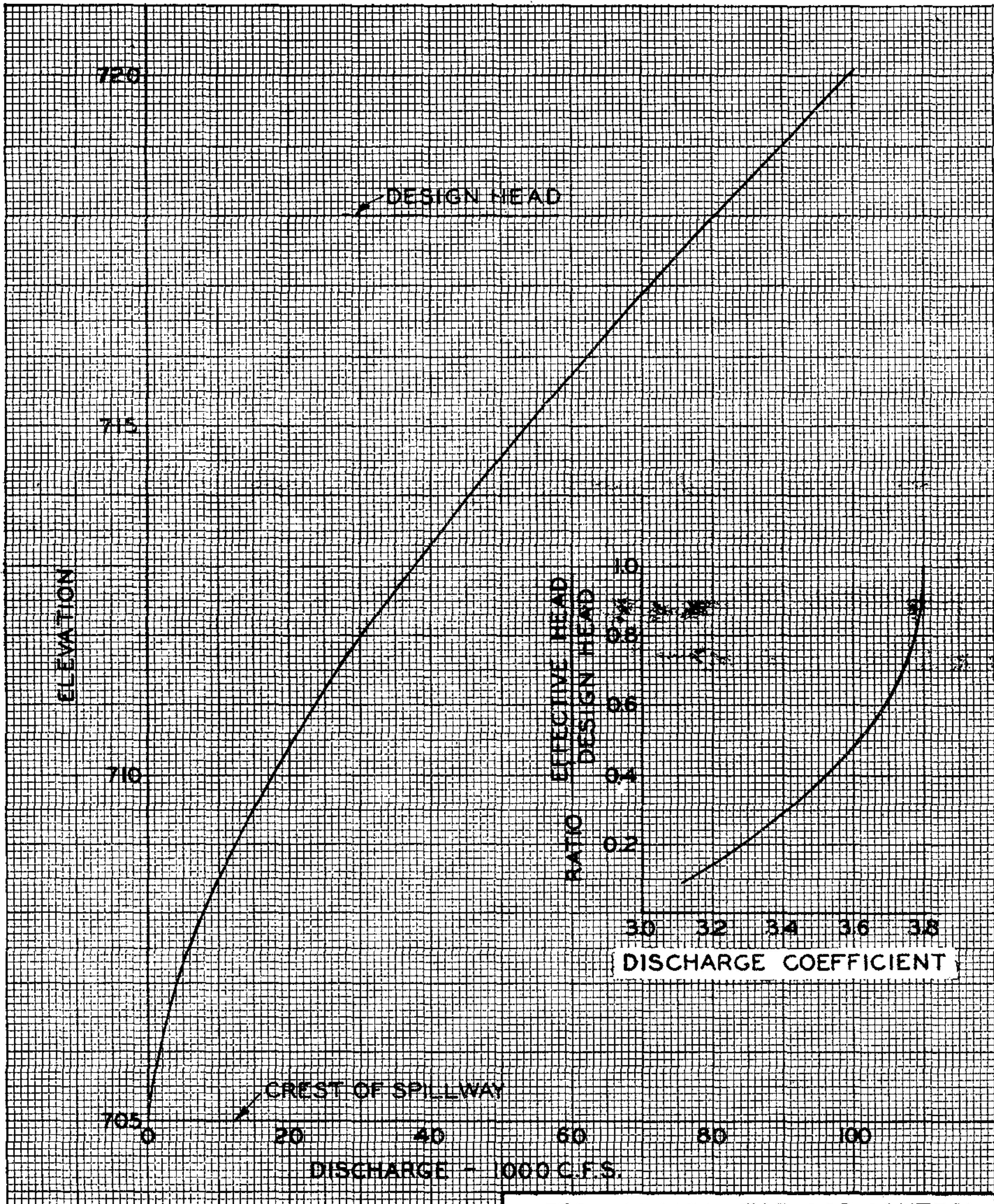
OCT. 1945 FILE NO. M16-12/66

PLATE III - 4



MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

SPILLWAY DISCHARGE CURVE

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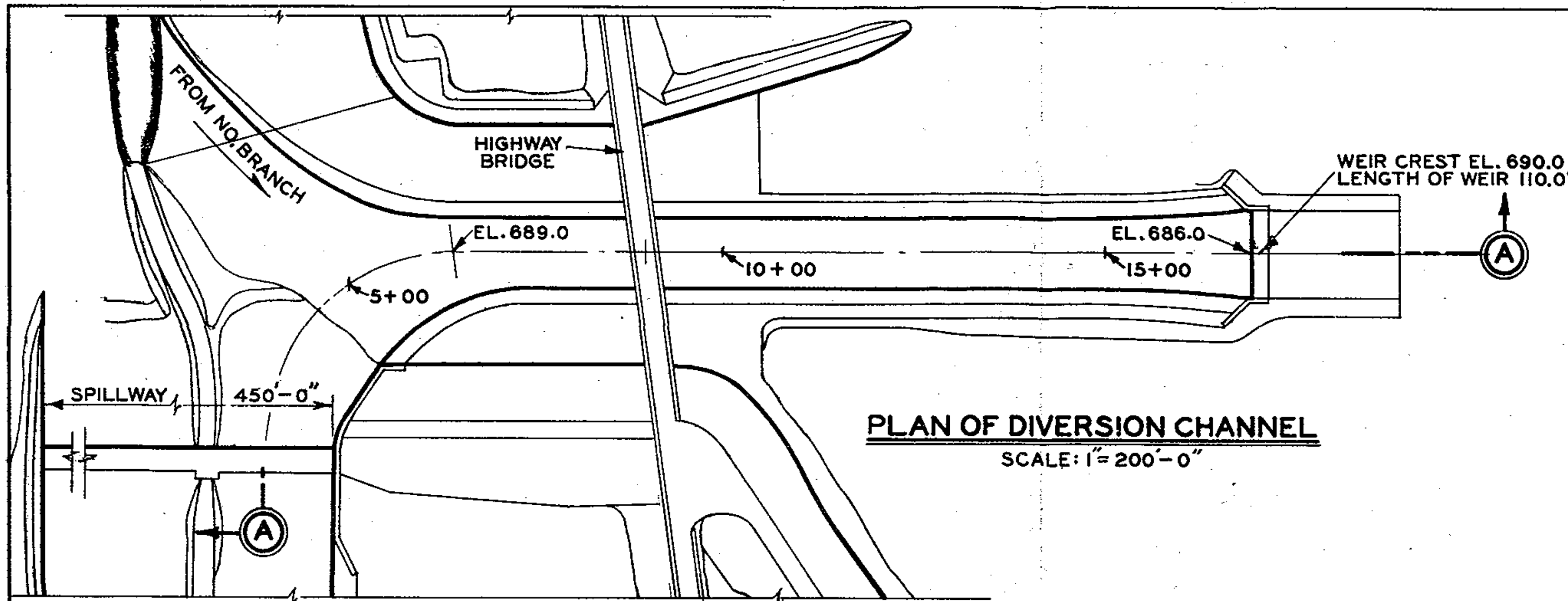


PLAN OF DIVERSION CHANNEL

SCALE: 1"= 200'-0"

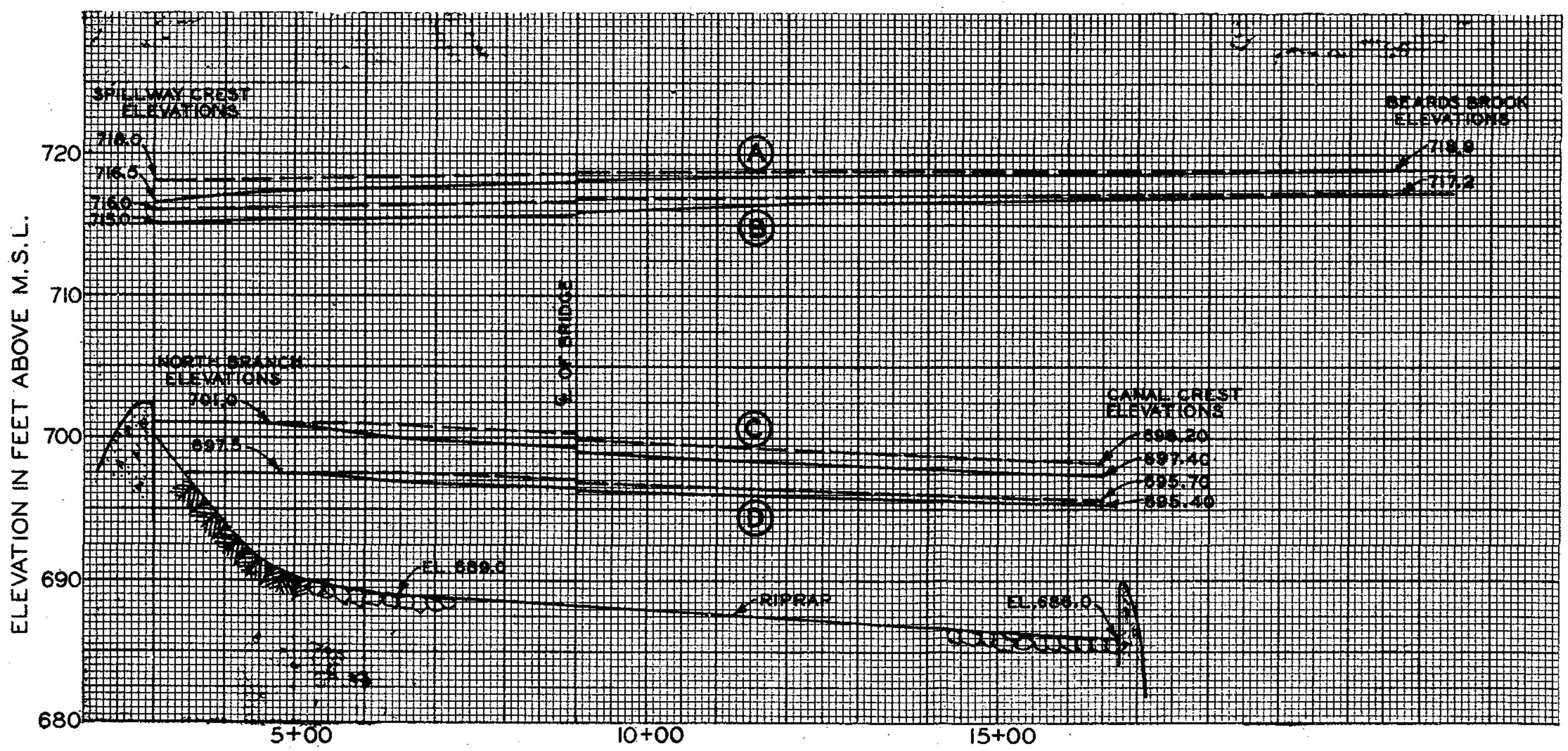
Notes:

1. Flood "A" is channel flow occuring at time of maximum reservoir stage during spillway design flood, assuming failure of Jackman Dam. Flow is from Beards Brook Reservoir towards North Branch. Channel flow = 44,000 C.F.S. Plate I - 22

2. Flood "B" is maximum channel discharge occuring during spillway design flood without failure of Jackman Dam. Flow is from Beards Brook Reservoir towards North Branch. Channel flow = 51,500 C.F.S. Plate I - 22

3. Flood "C" is canal flow producing maximum velocity in canal and occurs during computed spillway flood, assuming reservoir is empty at beginning of flood. Flow is from North Branch to Beards Brook Reservoir. Canal flow = 10,000 C.F.S. Plate I-23

4. Flood "D" is canal flow for reservoir design flood. Flow is from North Branch to Beards Brook Reservoir. Discharge = 5,000 C.F.S.



SECTION A - A

——— WATER SURFACE PROFILE
— — — ENERGY GRADIENT

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
BEARDS BROOK & NORTH BRANCH

DIVERSION CHANNEL
WATER SURFACE PROFILES

U.S. ENGINEER OFFICE — BOSTON, MASS.
OCT. 1945 — FILE NO. M16-12/68

APPENDIX IV

STRUCTURAL DESIGN

To accompany definite project report
dated November 1945

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX IV - STRUCTURAL DESIGN

CONTENTS

PLATES

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX IV

STRUCTURAL DESIGN

a. Introduction.- This appendix presents a detail description of the structures and improvements comprising the project; the results of the structural stability analysis made for the masonry structures; and an analysis of the selection of the gate type best suited and most economical for the installation in the intake tower. The masonry structures are designed in accordance with standard practice using working stresses and procedures prescribed in the Engineering Manual for Civil Works issued by the Office, Chief of Engineers. The final selection of each structure is governed by its suitability for the purpose involved and an economic study of the comparable types. Arbitrary loading assumptions are established, in all cases, following the criteria used on previously approved projects. The hydraulically operated sluice gates are selected for this installation because of their simplicity in operation and the lower cost of installation and maintenance.

b. Site Selection.- Engineering studies were made for various locations of the dam and spillway within the vicinity of the proposed dam site as indicated on Plates II-7 and IV-1. Layouts, estimates, and economic studies have been prepared for several types of dams in order to afford protection, from both the North Branch and Beards Brook, of Lower Hillsboro Village and the Town of Hillsboro and to reduce the flood stages in the Contoocook River. In addition to the control of the North Branch, it was necessary to consider the possibility of failure of the Jackman Dam located approximately 2500 feet upstream from the proposed spillway site. The location of the dam and spillway selected from the three alternates described in paragraph 15 proved to be the most economical while providing protection to the downstream towns from floods and the possible failure of Jackman Dam.

c. Consultants Conferences.- During the progress of the design, two Board of Consultants meetings were held in the Boston District Office and the selection of structures and features of design were discussed in detail. The first conference was held on 1 September 1944 and the following members were present:

Mr. W. H. McAlpine - Office, Chief of Engineers,
Washington, D. C.
Dr. Arthur Casagrande - Harvard University, Cambridge, Mass.,
Consultant
Mr. W. F. Uhl - Charles T. Main, Inc., Boston, Mass.,
Consultant.

The second conference was held on 14-15 December 1944, and the following members were present:

Mr. W. H. McAlpine, Office, Chief of Engineers,
Washington, D. C.
Mr. W. F. Uhl - Charles T. Main, Inc., Boston, Mass.,
Consultant
Mr. J. D. Justin - Philadelphia, Pennsylvania,
Consultant
Dr. Arthur Casagrande, Harvard University, Cambridge,
Mass., Consultant.

Informal discussions were held at various times with members of the Board of Consultants on various features of design.

A conference between representatives of the Boston District Office and the Office, Chief of Engineers was held on 30 May 1945 in the Office, Chief of Engineers, Washington, D. C. for the purpose of discussing the criteria and features of design. The representatives of the Office, Chief of Engineers afterward made an inspection tour of the dam site on 4 July 1945.

Prior to the completion of the final contract plans, one more meeting of the Board of Consultants will be held in order to review the contract plans.

d. Description of Structures.- (1) Embankment.- The proposed embankment, as shown on Plate IV-1, consists of a rolled earth embankment with dumped rock side slopes as shown in the cross section on Plate IV-3. The embankment has a crest length of approximately 4400 feet and a maximum height of 108 feet. A detailed analysis of design for the embankment is contained in Appendix II.

(2) Outlet Works.- (a) Controlled Outlet Works.- The controlled outlet works are located on Beards Brook and regulate the discharge therefrom. The outlet works consist of a reinforced concrete intake structure, a conduit through the embankment and stilling basin, sections of which

are shown on Plates IV-3, 4 and 5. The gate house contains two outlets 3'-0" x 7'-0" and one outlet 4'-0" x 7'-0", each of which contains an hydraulically operated sluice gate. The intake structure is connected with the embankment by a structural steel bridge. The gate chamber outlets converge into a single concrete conduit, 8 feet square which passes through the embankment and has a series of cut-off collars for control of seepage through the dam. The stilling basin consists of a reinforced concrete slab with reinforced concrete baffles and end sill founded on a drainage mat of 18 inches of gravel and 6 inches of sand. Gravity-type sections have been selected for the intake and stilling basin retaining walls in view of their greater durability under severe climatic conditions. The entire outlet works are founded on the underlying glacial till and the design analyses for the structures are shown on Plate IV-10.

(b) Uncontrolled Outlet Works.- The discharge from the North Branch is unregulated and passes through an ungated 4'-0" x 6'-0" rectangular outlet in the spillway section which is located across the North Branch.

(3) Spillway.- The spillway is located at the westerly end of the embankment and spans the North Branch. The most economical channel widths and weir length were computed in relation to the corresponding dam height required. The spillway is a low gravity-type concrete ogee section. The spillway has a crest length of 450 feet at elevation 705 and is founded on bedrock. Stability analyses for the spillway are shown on Plate IV-10. The approach channel is approximately 150 feet in length and has a width of 450 feet at the spillway which increases in width upstream until it joins the diversion canal. The channel floor is excavated on natural rock surface with the rock cut forming the southerly training wall and a gravity concrete wall on the northerly side. A pilot channel excavated in rock is located approximately in the center of the approach channel to direct the normal flow of North Branch to the uncontrolled outlet in the spillway. The floor of the channel slopes upstream to natural ground surface to provide drainage of the area after high water stages. The floor of the spillway discharge channel is at elevation 695 and slopes to the present ground surface. The channel is 450 feet wide at the spillway and gradually converges to a width of 340 feet in a length of approximately 400 feet before it discharges into North Branch. Sections of the channel not in excavated rock are protected by dumped riprap. A pilot channel is provided approximately in the center of the discharge channel for the normal discharge from North Branch which empties into the existing stream bed downstream from the spillway. The training wall ad-

jacent to the spillway on the north side of the channel is a gravity type concrete wall with the remaining earth cut slope protected by 5 feet of dumped rock. The natural rock cut on the southerly side of the channel forms the training surface for the spillway discharge.

(4) Dike.- An earth-filled dike with riprapped face on the channel side, a section of which is shown on Plate IV-2, is located on the westerly side of the diversion canal and the northerly side of North Branch between Jackman Reservoir and the diversion canal. This dike is provided for protection of the existing wood stave pipe line during periods of high water. The dike also provides additional protection to the property located northwesterly of the proposed dike.

(5) Diversion Canal and Diversion Channel.- A diversion canal and channel are located through a low ridge approximately 250 feet upstream from the spillway and connect the reservoir basin with the North Branch. (See Appendix III).

The diversion canal is a trapezoidal section having a bottom width of 90 feet and side slopes of 1 on 2 and is approximately 1100 feet long with the invert varying from elevation 689 near the spillway to elevation 686 at the control weir. (See Plate IV-2). A control weir, with crest elevation 690 and length of 110 feet is provided at the northerly end of the canal to control the velocity of flow in the canal. The discharge over the control weir will be dissipated in a dumped rock paved area before flowing overland to the reservoir pool.

The diversion channel is a trapezoidal section having a minimum bottom width of 300 feet with side slopes varying from 1 on 2 to 1 on 3, and is superimposed on the diversion canal. The bottom and side slopes of the diversion canal and channel are riprapped to prevent scour and erosion.

A new seven (7) span steel girder highway bridge constructed on concrete piers spans the diversion channel.

(6) Structural Stability Analyses.- The masonry structures are designed in accordance with standard methods and the results of the analyses are shown on Plate IV-10. The analyses for the intake structure are made with extreme high water in the reservoir and with the reservoir empty. The stability analysis for the upstream spillway wall is based on the lateral pressure of the earth retained, and water at spillway crest elevation shortly after a design flood in which the saturation line in the fill is above

crest elevation. The design of the stilling basin wall is based on the lateral pressure of the earth retained, and normal tailwater shortly after a design flood in which the saturation line in the fill is above normal tailwater. Uplift pressures applied to masonry structures founded on till are equal to 66 percent of the hydrostatic head over 100 percent of the base area and for structures founded on rock are full uplift over 100 percent of the area.

In view of the relatively light foundation loads resulting from the computations and the remoteness of any possible earthquake shock to the dam when the reservoir is full, the structures were not analyzed with additional earthquake loadings.

e. Selection of Gate Type.— Gate sizes were selected to provide hydraulic design discharge capacities without the necessity of operating with partially opened gates. (Appendix III). Types of gates and gate hoist having the necessary features required for the plan of operation are (1) Self-closing caterpillar gate with cable hoist; (2) Slide gate with motorized screw type gate stand; (3) Slide gate with hydraulic hoist. Valves of the types designed for operation under high heads at partial opening were not considered since this condition does not exist in the plan of reservoir operation.

Caterpillar gates with cable hoists are eliminated as being uneconomical in the small sizes required in this design. The arrangement of slide gate with motorized gate stand considered for this design would require the gate stand to be mounted on the operating floor above the maximum reservoir level in order to keep all electrical motors, controls and wiring out of the dampness of the gate well. This arrangement requires a 100 foot long gate stem with about ten intermediate support brackets which would be costly and require considerable maintenance. Each gate would require a separate gate stand and the space in which to house them. No condition of gate operation requires more than one gate to be operated simultaneously, therefore, the expense of the three gate stands is required only because of the arrangement. The low efficiency of the screw type hoist requires that the hoist torque motor be the equivalent of about 210 HP motor as against a 5 HP motor as required for the pump for a hydraulic hoist. The switch board, electrical wiring, and the electric service line from a point 1/2 miles from the gate house site, and the standby generator set all would have to be of capacity to serve the high amperage torque motor. The screw type hoist, because of its many mechanical and electrical parts, would require considerable maintenance.

The slide gate with hydraulic hoist was selected because of its simplicity of operation and maintenance, and the lower costs of providing (1) Electric power supply to the gate house structure; (2) Switchboard; (3) Controls; and, (4) Standby generator set. This arrangement, with only one operating and one standby motor driven oil pump and the small standby generator and switchboard provides a maximum amount of working space on the equipment floors and also makes possible the provision of a subfloor for storage space since the gate well will not be flooded as would be the case with either the caterpillar gate or the conventional slide gate without bonnet and cover.

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

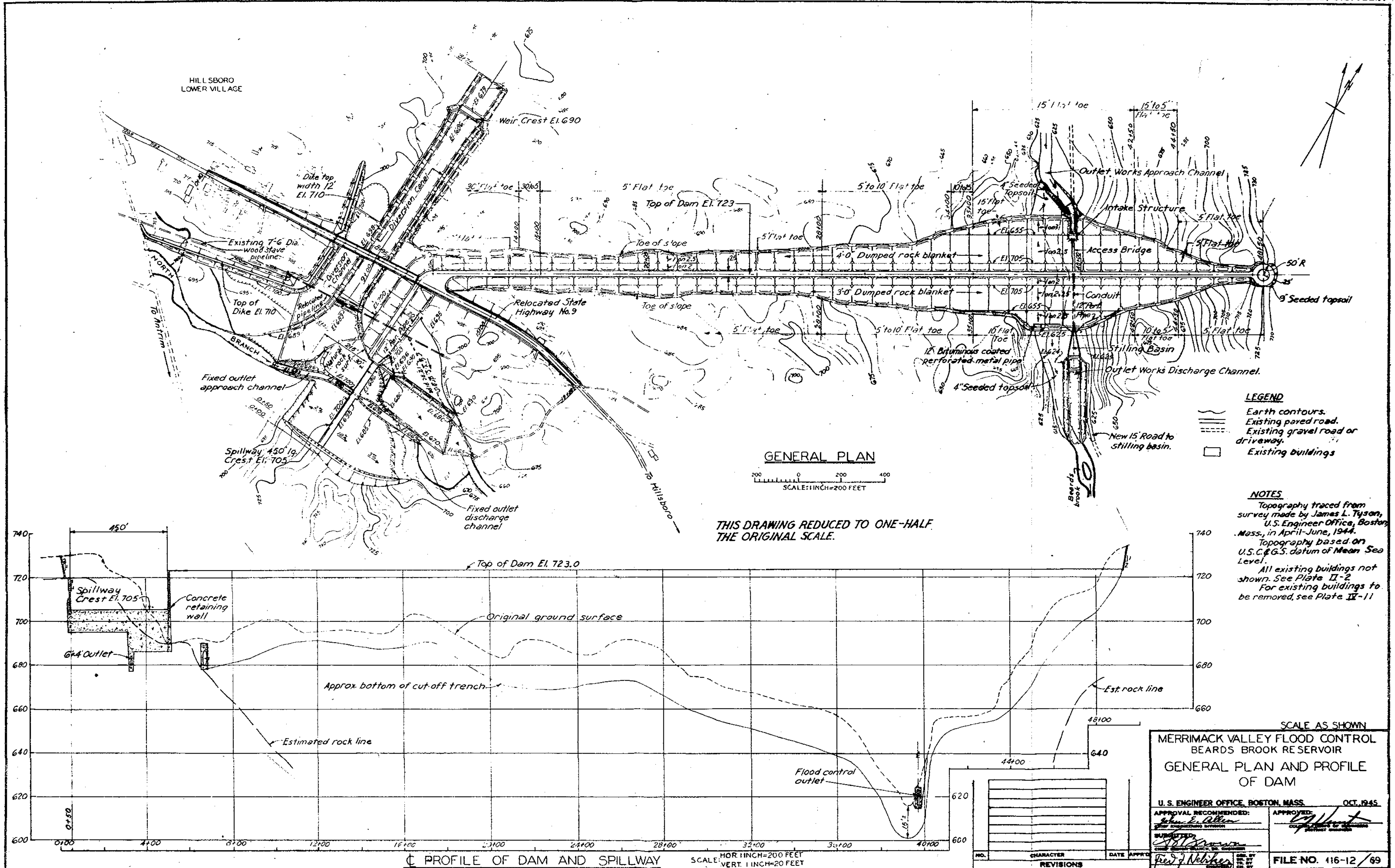


PLATE IV-1

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

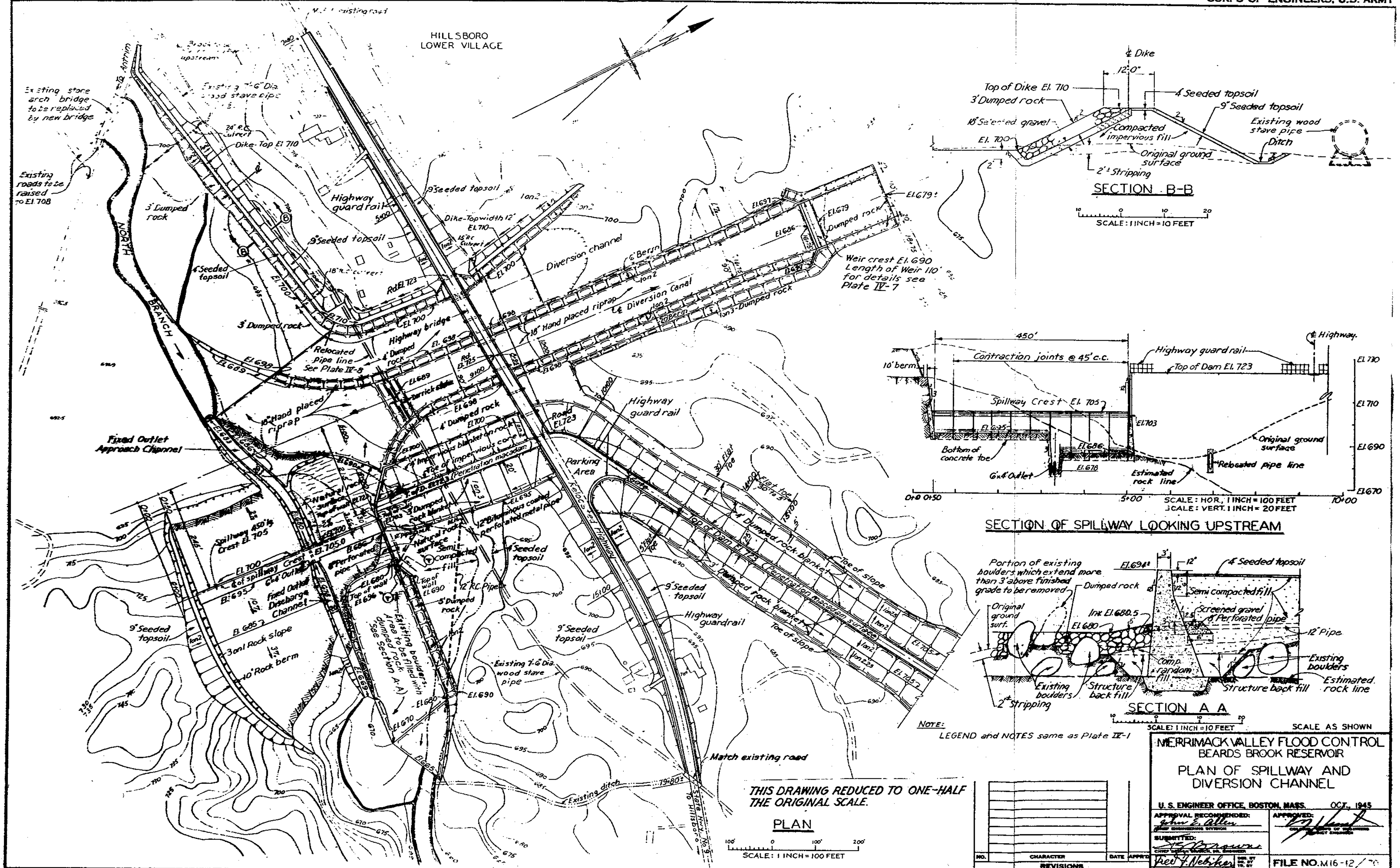
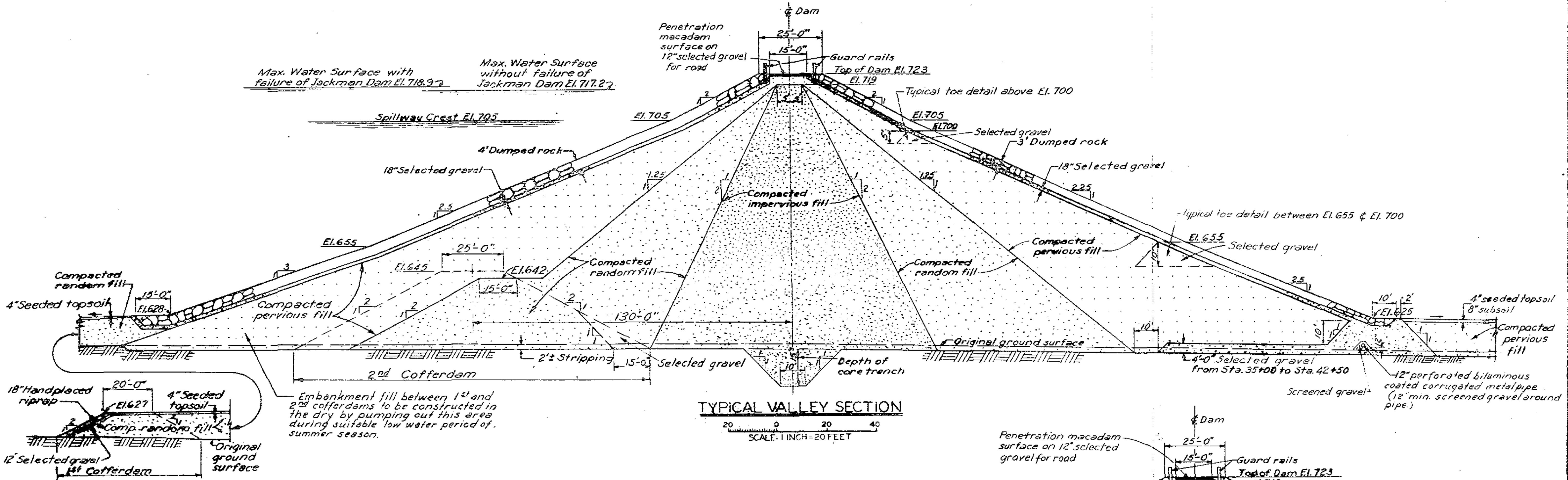
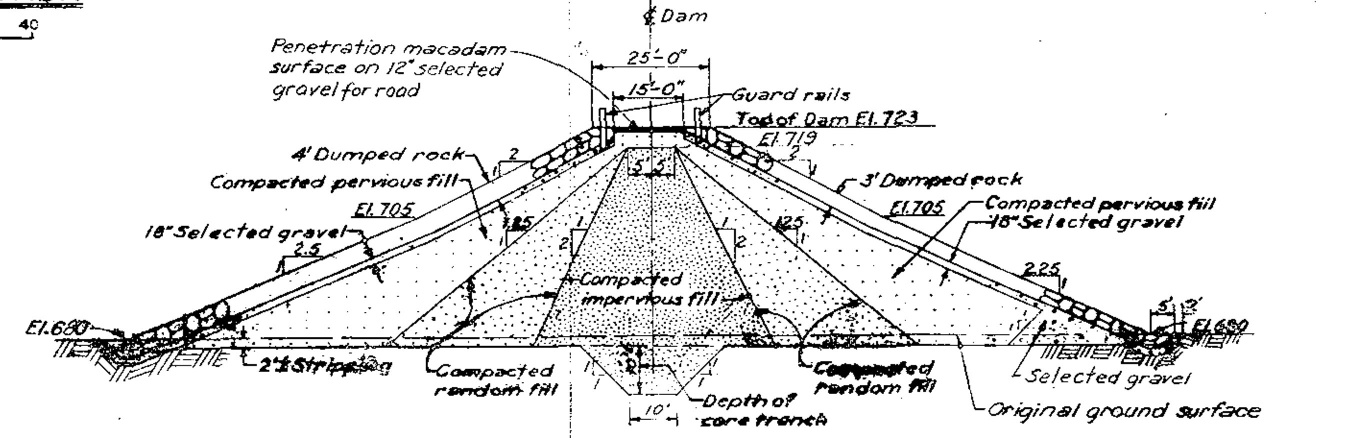


PLATE IV-2



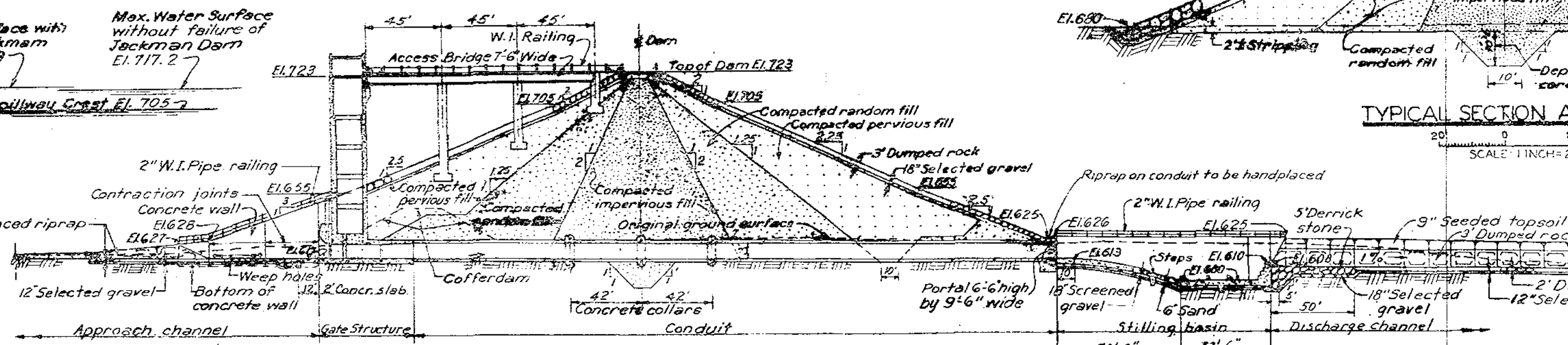
TYPICAL VALLEY SECTION

SCALE: 1 INCH = 20 FEET



TYPICAL SECTION ABOVE EL. 680

SCALE: 1 INCH = 20 FEET



SECTION THRU OUTLET WORKS

SCALE: 1 INCH = 40 FEET

Notes
Elevations are based on U.S.C.&G.S. datum, Mean Sea Level.

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

SCALE AS SHOWN

MERRIMACK VALLEY FLOOD CONTROL

BEARDS BROOK RESERVOIR

SECTIONS OF DAM AND OUTLET WORKS

U. S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945

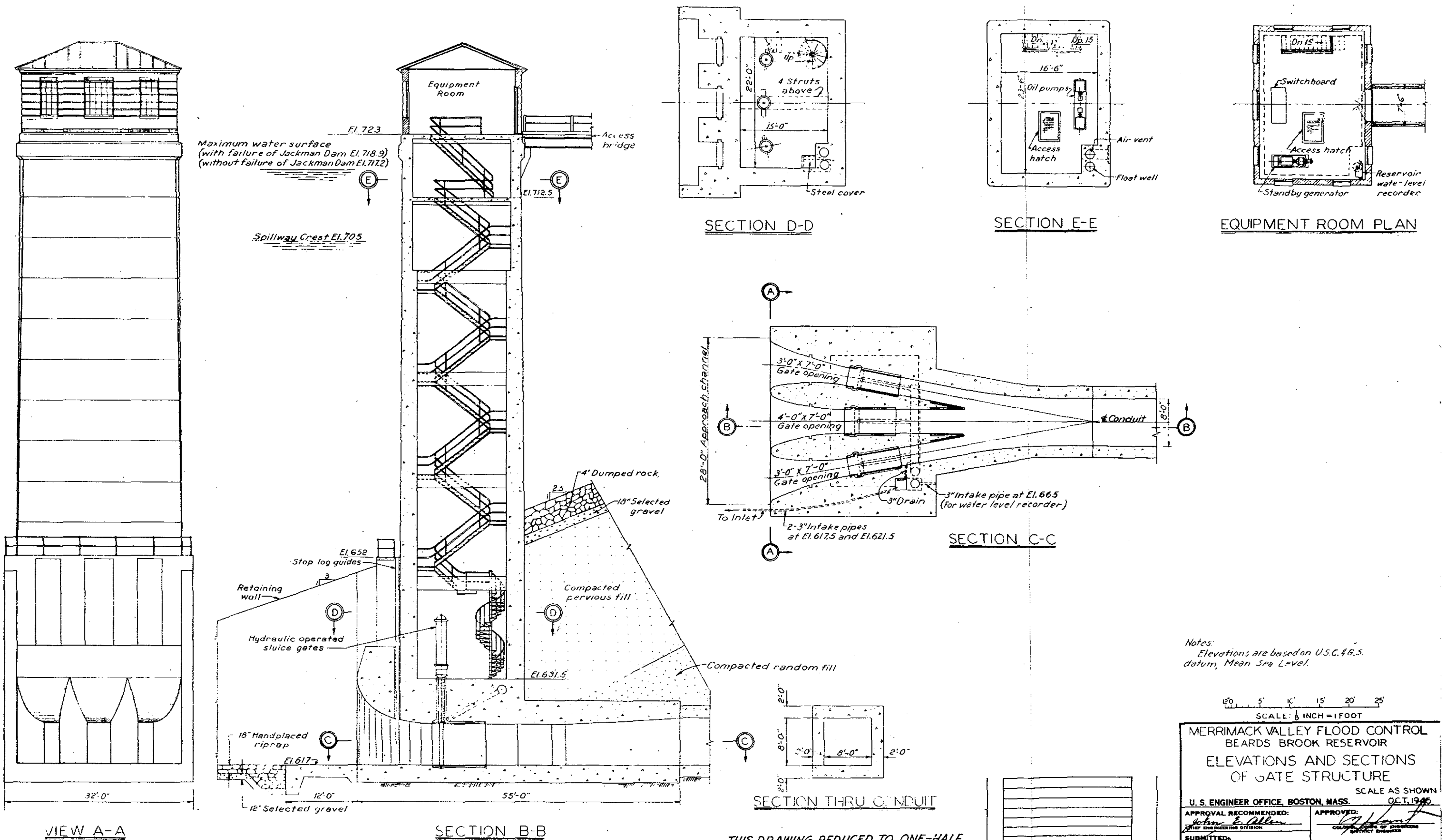
NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: COLONEL CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF DESIGN BRANCH

FILE NO. M16-12/71



THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

Notes:
Elevations are based on U.S.C.&G.S. datum, Mean Sea Level.

SCALE: 1/8 INCH = 1 FOOT

NO.	CHARACTER	DATE	APPR'D

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR

ELEVATIONS AND SECTIONS OF GATE STRUCTURE

SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1946

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: B. Brown, CHIEF DESIGN BRANCH, ENGINEER

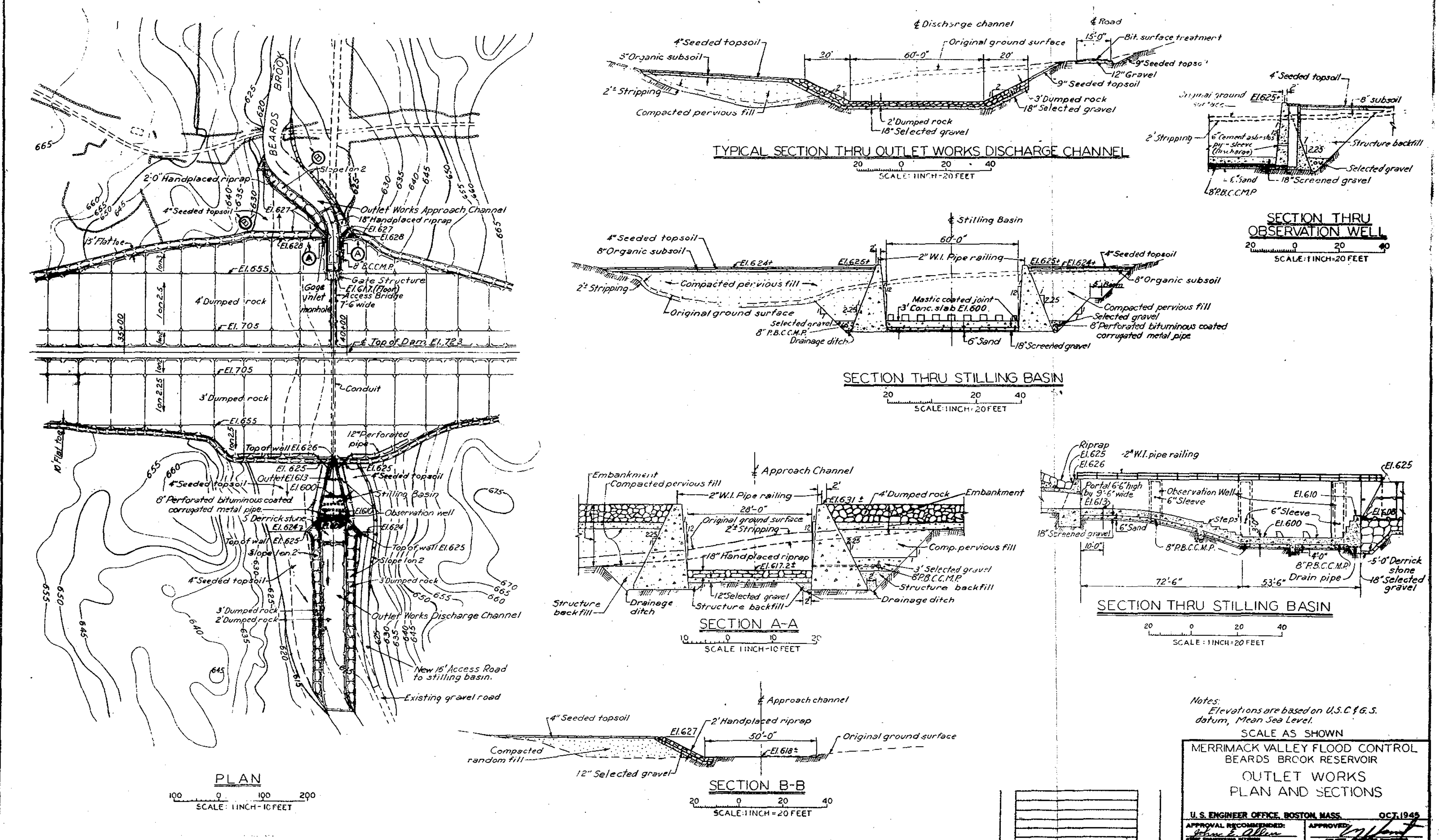
ENGINEER

FILE NO. M16-12/72

PLATE IV-4

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY



THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

Notes:
Elevations are based on U.S.C.&G.S. datum, Mean Sea Level.

SCALE AS SHOWN

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
OUTLET WORKS
PLAN AND SECTIONS

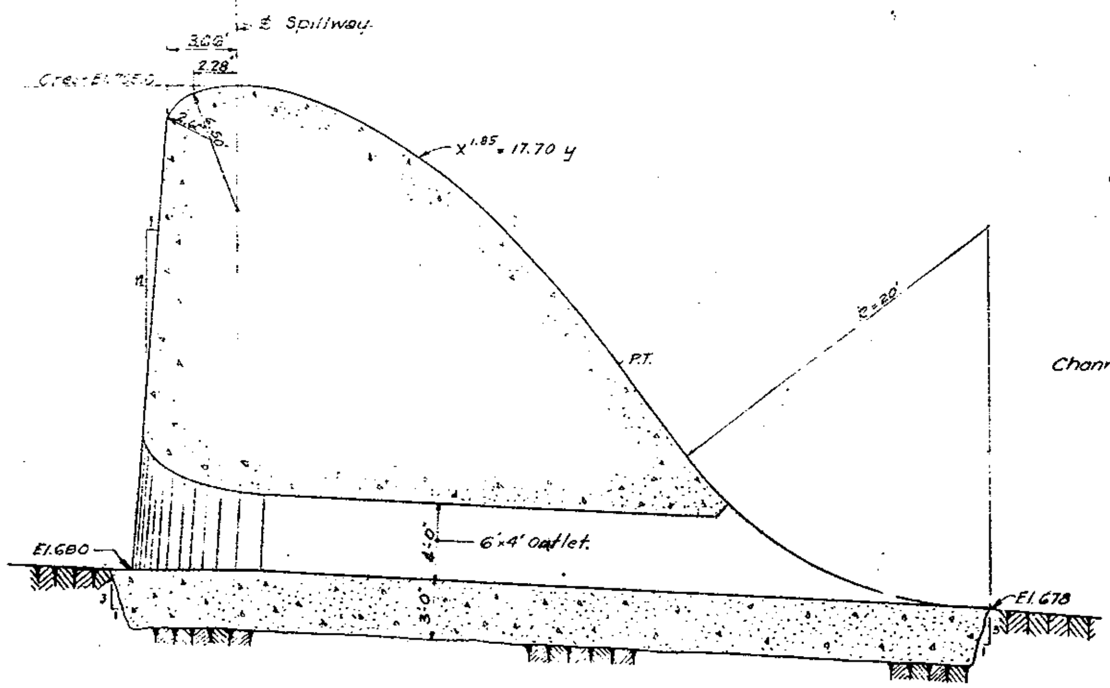
U.S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945

APPROVAL RECOMMENDED: John E. Allen
APPROVED:
SUBMITTED:

NO.	CHARACTER	DATE	APPR'D

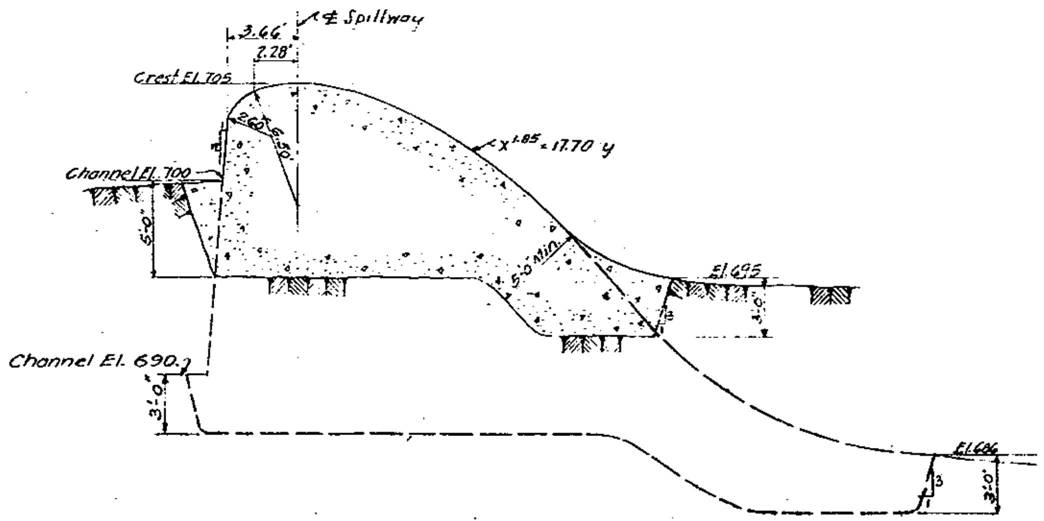
REVISIONS

FILE NO. M16-12/73



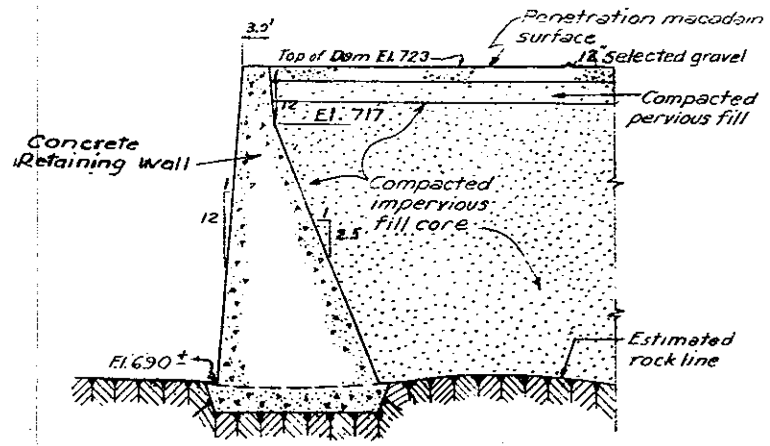
TYPICAL SECTION THRU FIXED OUTLET

SCALE: 1 INCH = 5 FEET



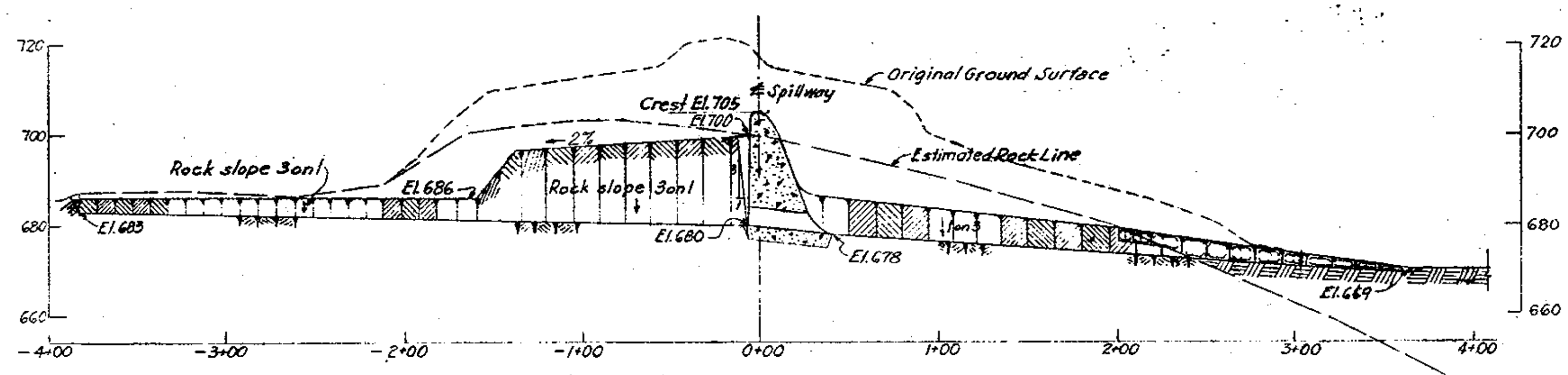
TYPICAL SPILLWAY SECTIONS

SCALE: 1 INCH = 5 FEET



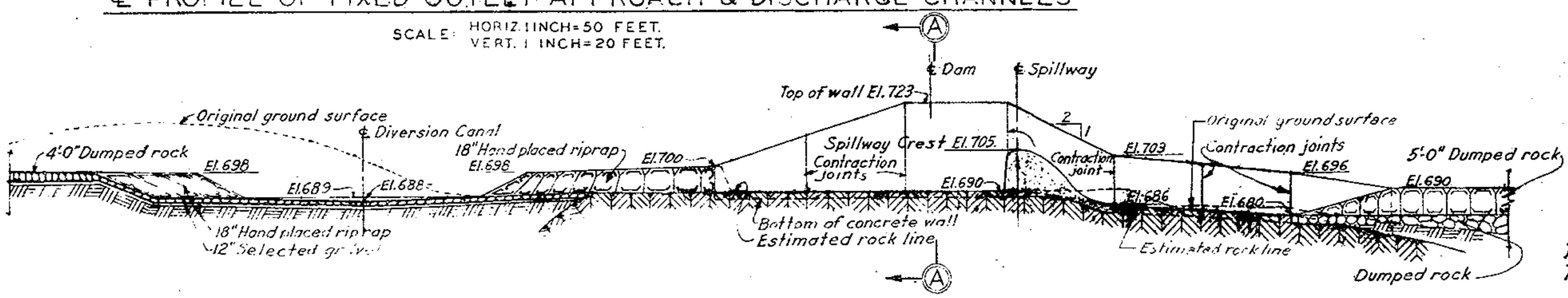
SECTION A-A

SCALE: 1 INCH = 10 FEET.



℄ PROFILE OF FIXED OUTLET APPROACH & DISCHARGE CHANNELS

SCALE: HORIZ. 1 INCH = 50 FEET. VERT. 1 INCH = 20 FEET.



PROFILE IN SPILLWAY CHANNELS ADJACENT TO RETAINING WALL

SCALE: 1 INCH = 30 FEET

Notes:
Elevations are based on U.S.C.&G.S. datum, Mean Sea Level.

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

SCALE AS SHOWN

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR

FIXED OUTLET CHANNEL PROFILES AND SECTIONS FOR SPILLWAY & WALL

U. S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: S. Braun, CHIEF DESIGN BRANCH

Fred J. Webster

NO.	CHARACTER	DATE	APPR'D

REVISIONS

FILE NO. M16-12/74

WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

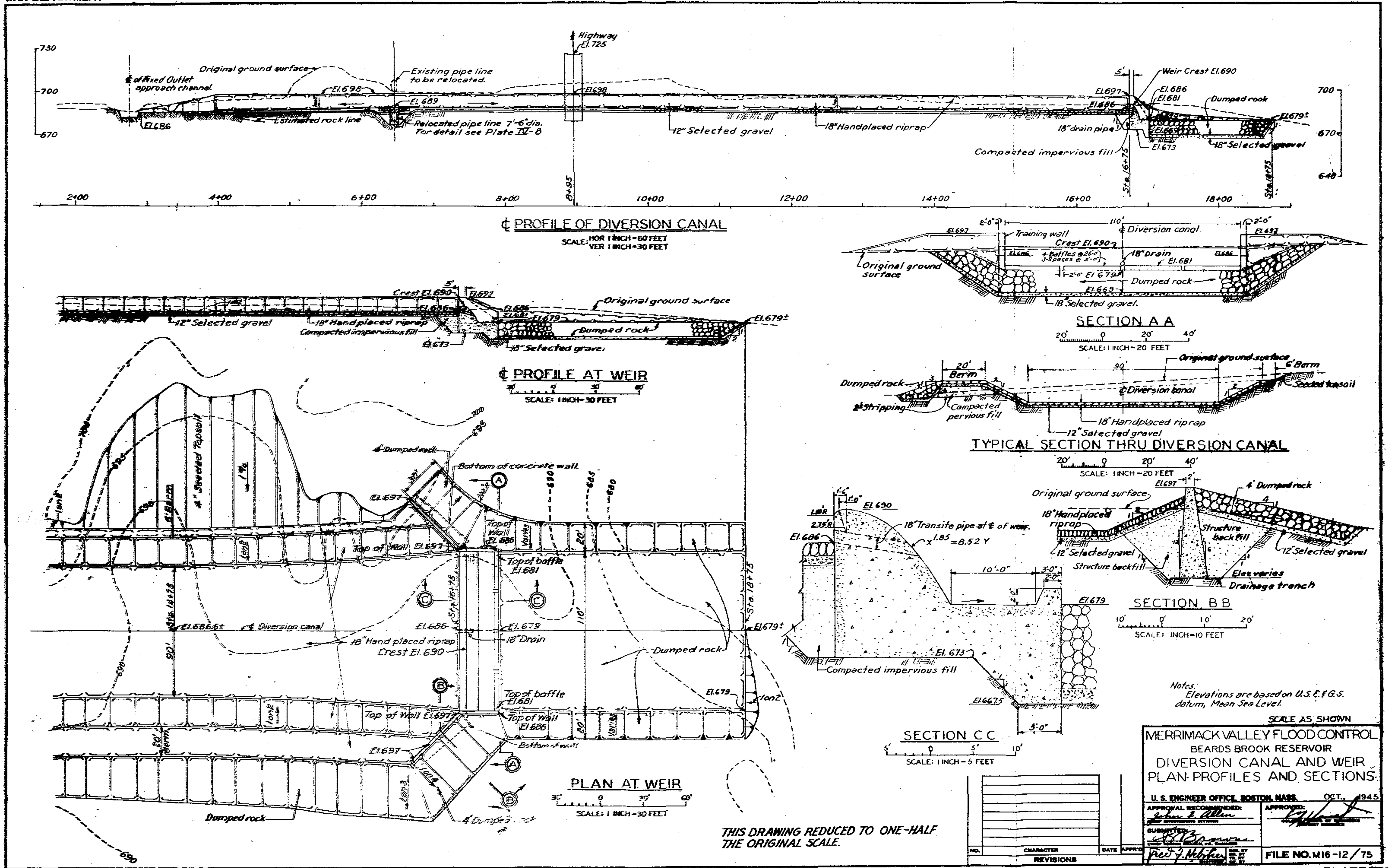
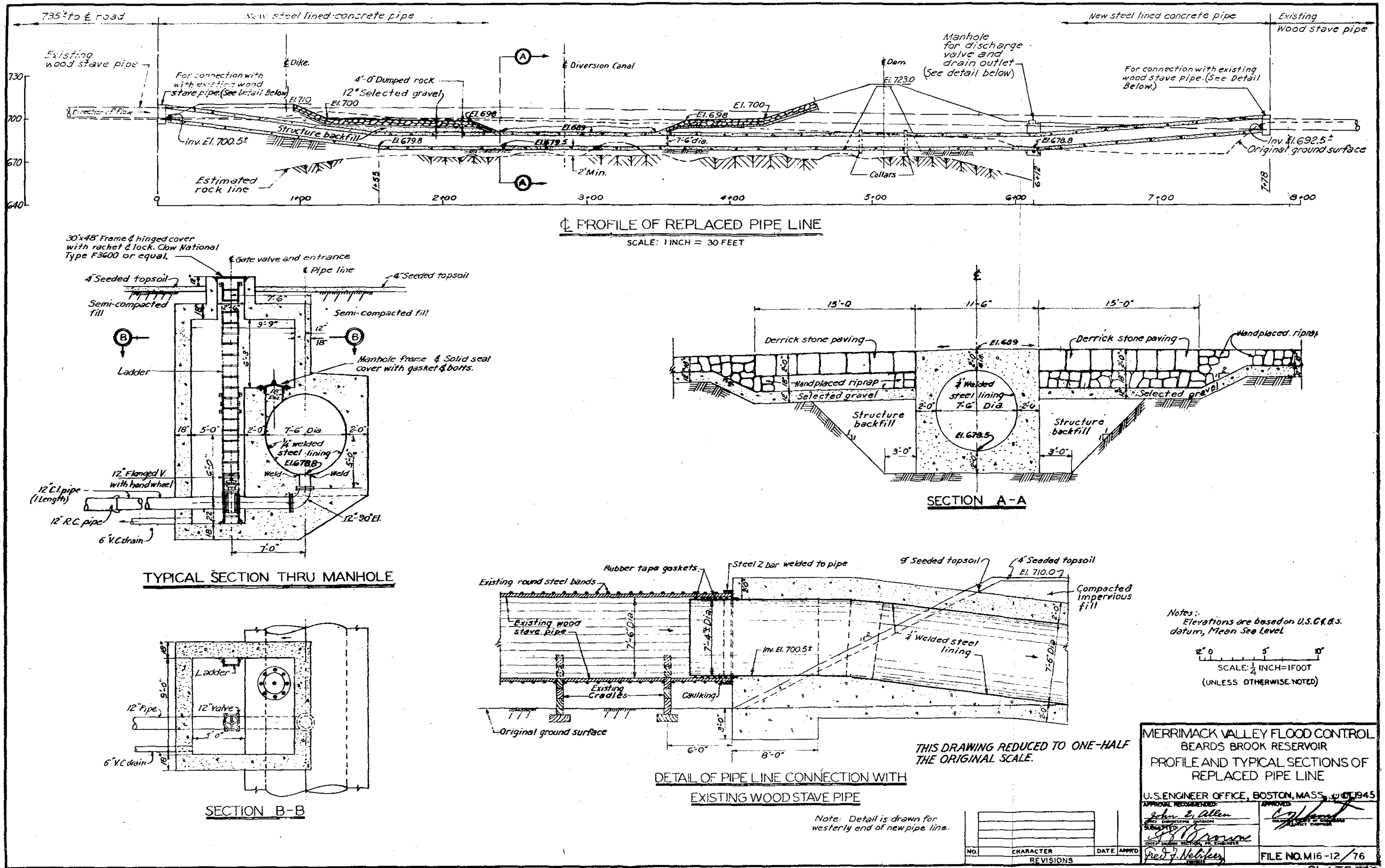
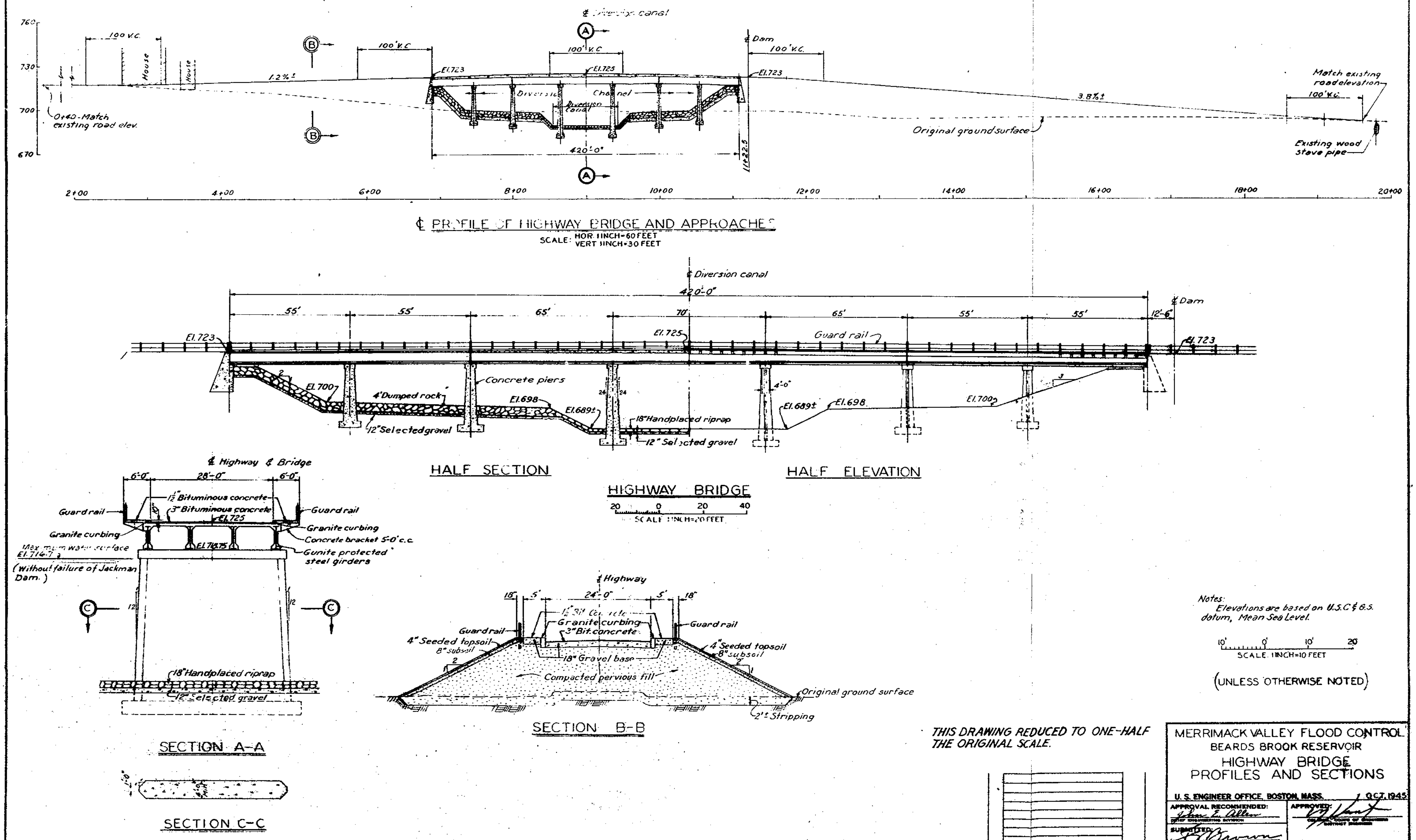


PLATE IV-7.



735± to ℄ road
New steel lined concrete pipe
New steel lined concrete pipe
Existing wood stave pipe
Existing wood stave pipe
For connection with with existing wood stave pipe (See detail Below)
℄ Dike.
4'-0" Dumped rock
12" Selected gravel
℄ Diversion Canal
Manhole for discharge valve and drain outlet (See detail below)
℄ Dam
El. 723.0
For connection with existing wood stave pipe. (See Detail Below.)
Direction of Flow
El. 710
El. 700
El. 700
El. 698
El. 698
El. 689
Structure backfill
Inv. El. 700.5±
El. 679.8
El. 679.5
7'-6" dia.
El. 678.8
Inv. El. 692.5±
Original ground surface
Estimated rock line
2' Min.
Collars
730
700
670
640
0
1+00
1+55
2+00
3+00
4+00
5+00
6+00
6+12
7+00
7+78
8+00
℄ PROFILE OF REPLACED PIPE LINE
SCALE: 1 INCH = 30 FEET
30"x48" Frame & hinged cover with rachet & lock. Clow National Type F3600 or equal.
℄ Gate valve and entrance
℄ Pipe line
4" Seeded topsoil
4" Seeded topsoil
Semi-compacted fill
Semi-compacted fill
Ladder
Manhole frame & Solid seal cover with gasket & bolts.
7'-6"
9'-9"
5'-0"
7'-6" Dia.
¼" welded steel lining
El. 678.8
Weld
Weld
12" Flanged V. with handwheel
12" C.I. pipe (1 Length)
12" R.C. pipe
6" V.C. drain
12"-90° El.
7'-0"
TYPICAL SECTION THRU MANHOLE
Ladder
12" Pipe
12" Valve
5'-0"
9'-0"
6" V.C. drain
SECTION B-B
15'-0
11'-6"
15'-0"
Derrick stone paving
Derrick stone paving
El. 689
Handplaced riprap
Handplaced riprap
Selected gravel
Selected gravel
¼" Welded steel lining 7'-6" Dia.
El. 679.5
Structure backfill
Structure backfill
3'-0"
3'-0"
SECTION A-A
Rubber tape gaskets
Steel Z bar welded to pipe
9" Seeded topsoil
4" Seeded topsoil
El. 710.0
Existing round steel bands
Compacted impervious fill
Existing wood stave pipe
7'-6" Dia.
7'-4½" Dia.
Inv. El. 700.5±
¼" Welded steel lining
7'-6" Dia.
Existing Cradles
Caulking
Original ground surface
6'-0"
8'-0"
DETAIL OF PIPE LINE CONNECTION WITH EXISTING WOOD STAVE PIPE
Note: Detail is drawn for westerly end of new pipe line.
THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.
Notes:- Elevations are based on U.S.C.&G.S. datum, Mean Sea Level
SCALE: ¼ INCH = 1 FOOT
(UNLESS OTHERWISE NOTED)
NO.
CHARACTER
DATE
APPR'D
REVISIONS
MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
PROFILE AND TYPICAL SECTIONS OF REPLACED PIPE LINE
U.S. ENGINEER OFFICE, BOSTON, MASS., OCT. 1945
APPROVAL RECOMMENDED:
APPROVED:
SUBMITTED:
FILE NO. M16-12/76
PLATE IV-8



MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
HIGHWAY BRIDGE
PROFILES AND SECTIONS

U. S. ENGINEER OFFICE, BOSTON, MASS. 1 OCT. 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

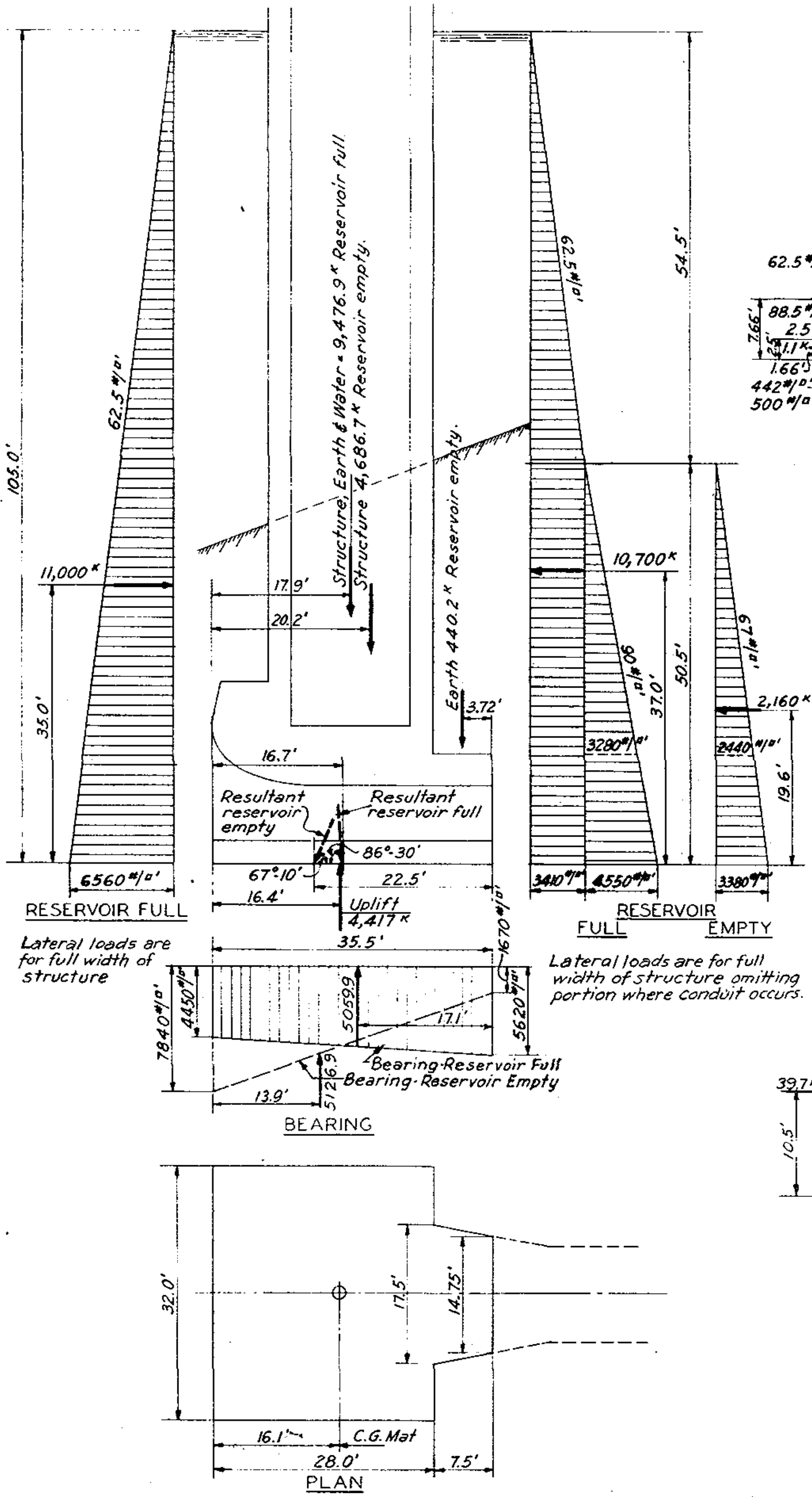
APPROVED: COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: CHIEF DESIGN BRANCH, ENGINEERING DIVISION

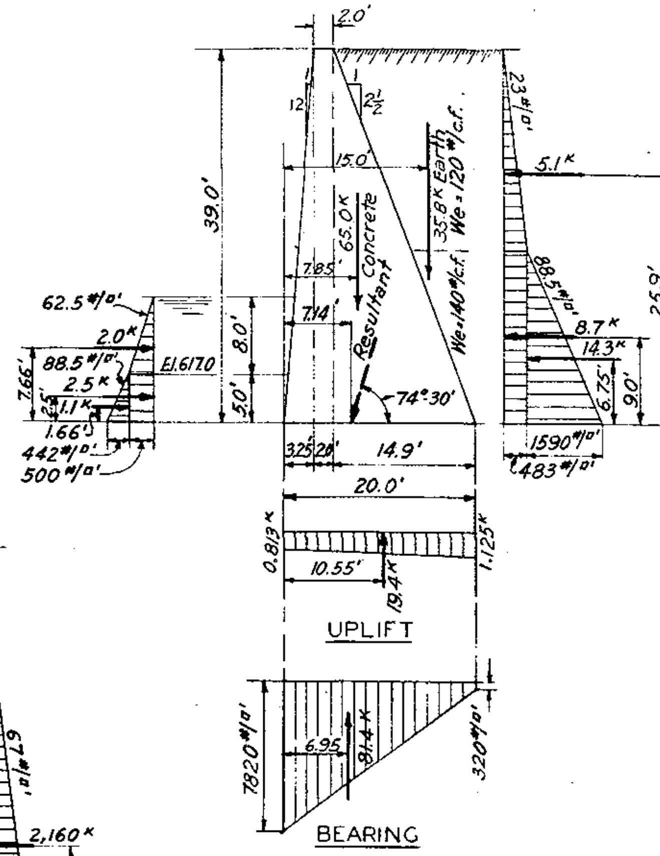
Fred J. Nelsiker

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

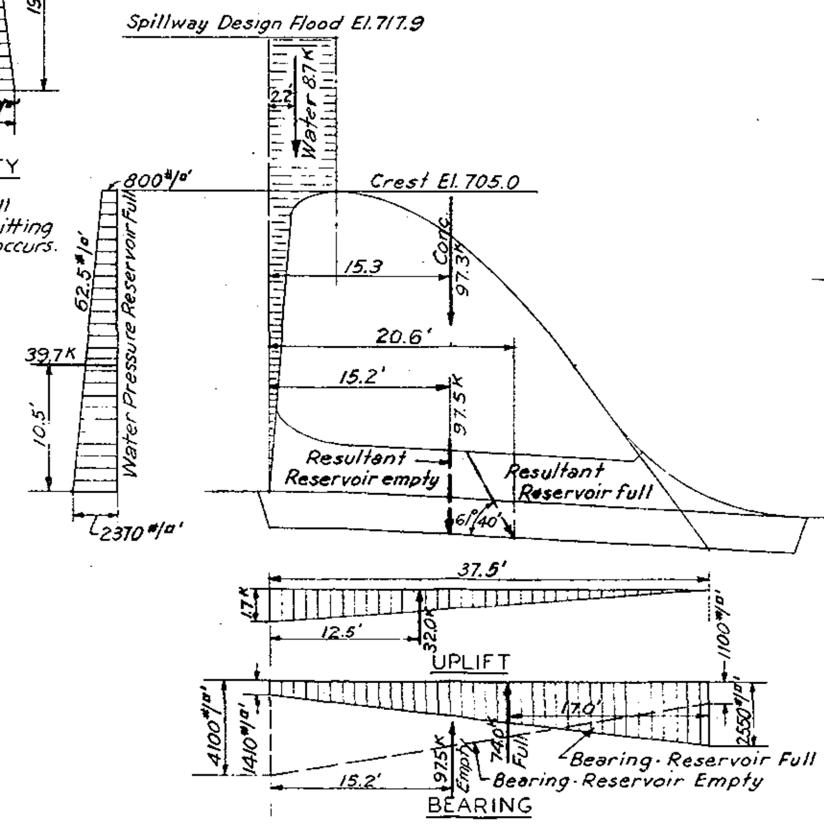
FILE NO. M16-12/77



GATE STRUCTURE

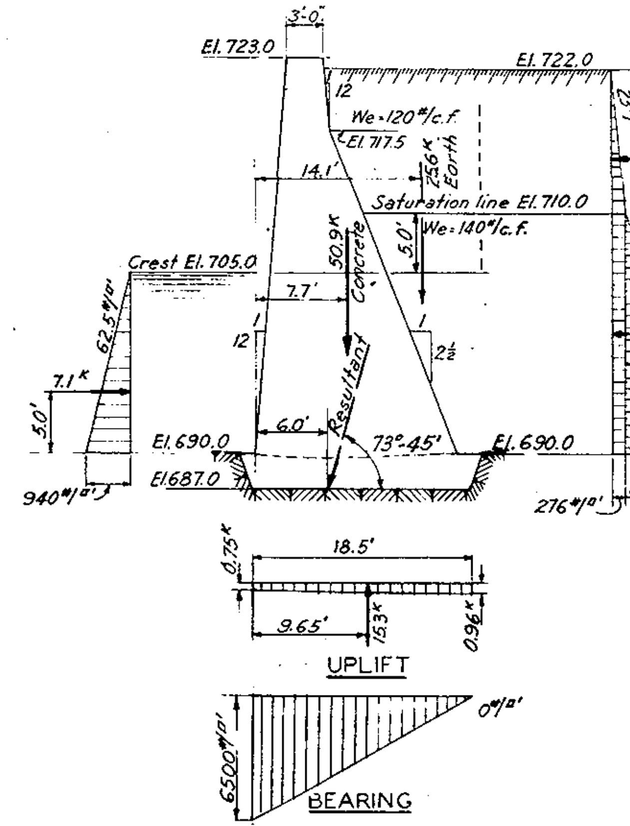


INTAKE APPROACH WALL



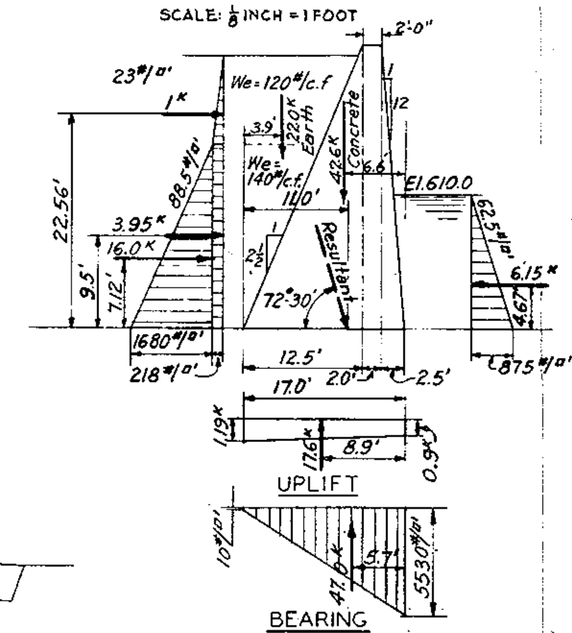
SPILLWAY AT FIXED OUTLET

SCALE: 1/8 INCH = 1 FOOT

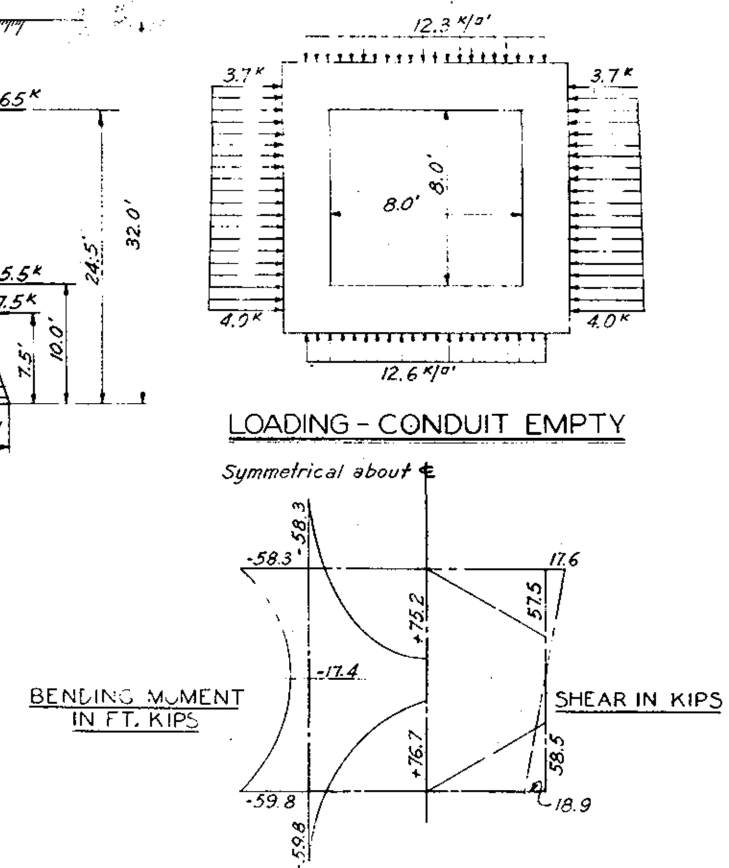


SPILLWAY CHANNEL WALL

SCALE: 1/8 INCH = 1 FOOT



STILLING BASIN WALL



CONDUIT

SCALE: 1/4 INCH = 1 FOOT

DESIGN ASSUMPTIONS AND GENERAL NOTES

Weight of concrete	150 #/cu. ft.
Weight of water	62.5 #/cu. ft.
Weight of shell material (dry)	120 #/cu. ft.
Weight of shell material (saturated)	140 #/cu. ft.
Weight of core material (dry)	140 #/cu. ft.
Weight of core material (saturated)	150 #/cu. ft.
Horizontal pressure of shell material (dry)	23 #/sq. ft.
Horizontal pressure of shell material (saturated)	26 #/sq. ft.
Horizontal pressure of core material (dry)	33 #/sq. ft.
Horizontal pressure of core material (saturated)	36 #/sq. ft.

Uplift between rock and concrete - 66% over 100% of area
Uplift between earth and concrete - Full uplift over 100% of area.

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

SCALE: 1 INCH = 10 FEET
(UNLESS OTHERWISE NOTED)

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
STABILITY ANALYSES
CONCRETE STRUCTURES

U.S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

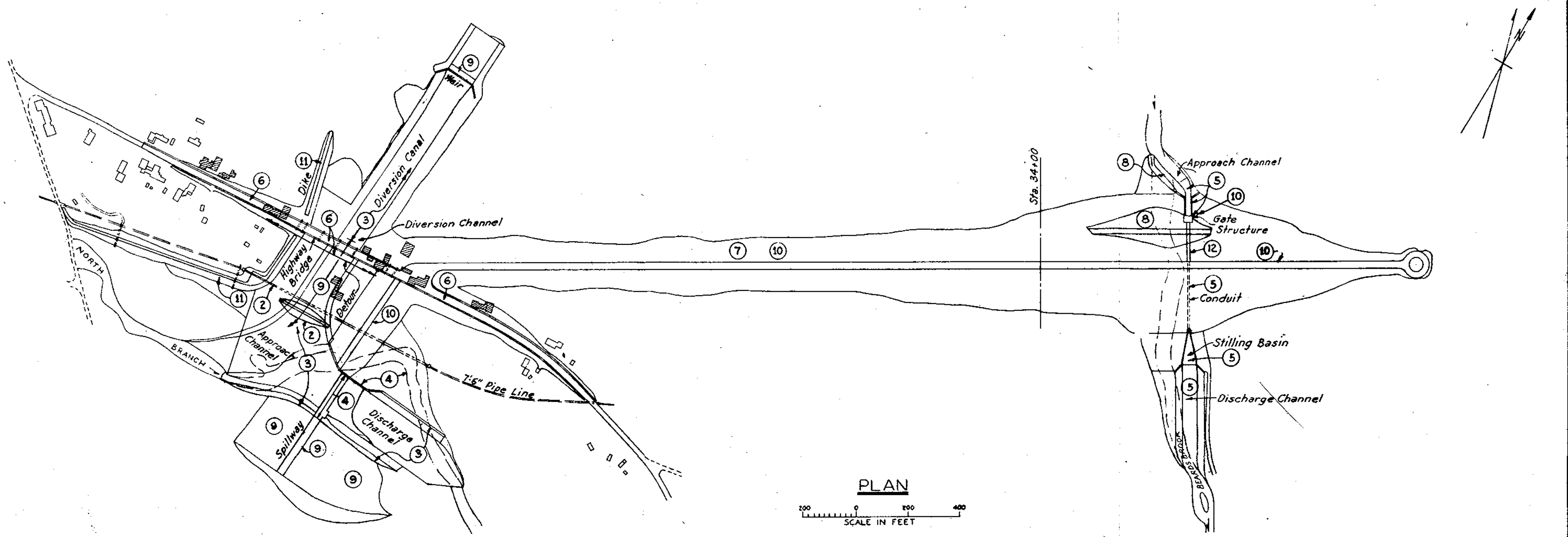
APPROVED: [signature], COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: [signature] Brown, CHIEF DESIGN BRANCH, PR. ENGINEER

Elmer P. Lindsay, ENGINEER

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

FILE NO. M16-12/78



SCHEDULE OF CONSTRUCTION OPERATIONS

FIRST SEASON		SECOND SEASON		THIRD SEASON	
DESCRIPTION OF OPERATIONS	APPROXIMATE QUANTITY CU. YARDS	DESCRIPTION OF OPERATIONS	APPROXIMATE QUANTITY CU. YARDS	DESCRIPTION OF OPERATIONS	APPROXIMATE QUANTITY CU. YARDS
(1) Remove existing buildings ___, Clear and strip site.	Clearing....60 Acres Stripping....145,000	(8) Divert Beards Brook thru conduit, construct cofferdams across Beards Brook.	Excavation....2,000 Fill........27,000	(12) Construct access bridge to gate house. Final grading; place subsoil and topsoil. Surface for road on embankment and road to stilling basin.	Lump Sum Fill 15,000 9,000 Sq.Yds.
(2) Construct 7'-6" diameter pipe line and adjacent cofferdam.	Excavation.... 7,500 Fill........5,000 Concrete......2,600	(9) Complete the spillway, spillway approach and discharge channels, diversion channel and diversion canal.	Excavation..155,000 Fill........50,000 Concrete.... 4,000		
(3) Excavate diversion canal and portion of diversion channel. Construct portion of spillway approach and discharge channels.	Excavation...98,000 Fill........13,000	(10) Complete the intake structure and embankment.	Excavation....25,500 Fill.......844,000 Concrete.......700		
(4) Construct portion of spillway to El. 695, entire retaining wall and filled area north of spillway discharge channel.	Excavation....2,500 Fill........40,000 Concrete......5,500	(11) Construct dike along diversion channel	Fill..........16,000		
(5) Construct conduit; stilling basin and walls; approach channel walls; intake structure to El. 650; portion of approach and discharge channels.	Excavation....23,000 Fill........30,000 Concrete......6,400				
(6) Construct highway bridge and approaches; detour, using portion of existing state highway	Excavation.....1,000 Fill........28,000 Concrete......2,600				
(7) Construct embankment to El. 705 from Sta. 8+30 to Sta. 34+00.	Excavation...21,500 Fill.......346,000				

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE

SCALE AS SHOWN

NO.	CHARACTER	DATE	APPR'D

REVISIONS

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR

SCHEDULE OF CONSTRUCTION OPERATIONS

U. S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945.

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: Brown, CHIEF DESIGN BRANCH

Fred J. Nelikes

FILE NO. M16-12/79

APPENDIX V

HYDROELECTRIC POWER

To accompany definite project report
dated November 1945

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX V – HYDROELECTRIC POWER

CONTENTS

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX V

HYDROELECTRIC POWER

a. Introduction.- This appendix presents the results of studies made of the feasibility of developing Beards Brook Reservoir in the interests of flood control and storage for stream regulation. The studies were made pursuant to the recommendation of the Federal Power Commission that a portion of the flood control capacity be used for stream regulation storage at some future time, which recommendation was supported by the Board of Engineers for Rivers and Harbors in the 9th Indorsement to a letter dated 8 November 1943, Subject: "Reservoir Plans for the Contoocook Basin, New Hampshire". The results of the studies indicate that multiple-purpose development cannot be economically justified.

b. Studies.- The storage capacity of 35,000 acre-feet represents the maximum obtainable storage without relocating Hillsboro Lower Village and is the minimum storage required for flood control purposes (See Appendix I). It is possible, however, that at some future time a portion of the flood control storage capacity of Beards Brook Reservoir may be re-allocated to another reservoir farther upstream and that an equivalent portion of the storage capacity in Beards Brook Reservoir may be used for stream regulation, with or without hydroelectric development at the site. An economic study has, therefore, been made and the results summarized in Table A accompanying this appendix.

c. Estimated Cost.- Re-allocation of the proposed flood control storage capacity with spillway elevation 705 for multiple-purpose use, undertaken as a second stage project in the future, would involve additional clearing of approximately 500 acres of wooded land. The principal items of construction involved in the second stage development would be clearing for Plans I and III, and clearing and construction of power generation facilities for Plan II. The cost of the hydroelectric power development is based on a plant with 3,000 K.W. installed capacity at a 20 percent load factor, operating with an average net head of 90 feet.

d. Basis of Cost Analysis.- The evaluation of power benefits to be derived from a hydroelectric plant located at the proposed dam site and the existing downstream hydroelectric installations with the reservoir operated to increase the low water flows, has been based on the estimated costs for the production of equivalent power by a steam plant located at a load center such as Manchester, New Hampshire. The construction cost of such a steam plant is estimated to be $102. per K.W., and the annual cost, consisting of the fixed charges and allowance for operation and maintenance, is estimated to be $17.50 per kilowatt per year. This amount is the value assigned per kilowatt of dependable prospective hydroelectric capacity. The value of energy output, based on the current cost of coal fuel at Manchester, is estimated at about 3 Mills per kilowatt hour. An allowance of 10 percent has been made for losses in transmission.

_e_. _Conclusion_.- The studies of the benefits to be derived from the development of a multiple-purpose reservoir instead of a flood control reservoir indicate that it can not be economically justified (as illustrated in the accompanying Table). The ratio of annual benefits to carrying charges for flood control and stream regulation storage is .25 and for stream regulation and hydroelectric power generation at site is 0.48. These low ratios do not warrant making any provision in the initial construction for future power installation.

COST ANALYSIS FOR STREAM REGULATION STORAGE AND POWER INSTALLATION AT SITE

1. Reservoir Data.-

	Plan 1 Flood Control and Stream Regulation Storage	Plan 2 Stream Regulation Storage with Power Installation at Site	Plan 3 Stream Regulation Storage Only
Total Storage Capacity	35,000 A.F., El.705	35,000 A.F., El.705	35,000 A.F., El.705
Flood Control Storage	20,000 A.F., El.692-El.705	None	None
Stream Regulation or Power Storage	13,000 A.F., El.545-El.692	28,000 A.F., El.660-El.705	30,000 A.F., El.655-El.705
Storage at Maximum Draw-down	2,000 A.F., El.545	7,000 A.F., El.660	5,000 A.F., El.655
Water Utilization Factor	80%		80%

2. Estimated Cost.-

	Plan 1 Charged to Flood Control	Plan 1 Charged to Stream Regulation	Plan 2	Plan 3
Reservoir Costs (Including Buildings and Road Relocations)	$ 139,000	$ 105,000	$ 244,000	$ 244,000
Construction Costs	1,669,000	1,251,000	2,920,000	2,920,000
Reservoir Clearing and Relocation of Water Supply Pipe Line	12,000	120,000	156,000	156,000
Subtotal	$1,820,000	$1,476,000	$3,320,000	$3,320,000
Cost of Power Installation for 3,000 K.W. Capacity	-	-	530,000	-
TOTAL ESTIMATED COST	$1,820,000	$1,476,000	$3,850,000	$3,320,000
Cost per Acre Foot of Stream Regulation Storage	-	$98.40	$94.90	$94.90

COST ANALYSIS FOR STREAM REGULATION STORAGE AND POWER INSTALLATION AT SITE
(Cont'd.)

	Plan 1 Flood Control and Stream Regulation Storage		Plan 2 Stream Regulation Storage with Power Installation at Site	Plan 3 Stream Regulation Storage Only
3. Annual Carrying Charges.-	Charged to Flood Control	Charged to Stream Regulation		
	$81,000	$66,000	$183,000	$149,000
4. Annual Power Benefits.-				
(a) Power Installation at Site:				
(1) 3,000 KW @ $17.50			$ 52,500	
(2) 4,000,000 KWHS @ 3 mills			$ 12,000	
(b) Increased Output at Developed Hydro-Electric Plants Downstream (Total Head - 138 Feet)				
(1) Increase in Prime Peaking Capacity @ $17.50 per KW		700 KW-$12,250	900 KW-$ 15,750	1,000 KW-$ 17,500
(2) Increase in Average Annual Energy Output @ 3 Mills per KW		1,300,000 KWHS-$ 3,900	2,600,000 KWHS-$ 7,800	2,800,000 KWHS-$ 8,400
		$16,150	$ 88,050	$ 25,900
5. Ratio of Annual Benefits to Annual Carrying Charges.-		.25	.48	.17

APPENDIX VI

RELOCATIONS

To accompany definite project report
dated November 1945

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX VI - RELOCATIONS

CONTENTS

PLATES

DEFINITE PROJECT REPORT
BEARDS BROOK RESERVOIR

APPENDIX VI.

RELOCATIONS

a. Introduction.- The construction of Beards Brook Reservoir necessitates raising one first class state highway which now passes through the proposed dam site; raising a short stretch of a secondary gravel road and replacing the existing bridge with a new bridge; and the improvement of an existing access road to a cemetery. The raising of the roads also requires the raising of accompanying power and telephone lines. There are no railroads or cemeteries within the proposed reservoir basin. The Public Service Company of New Hampshire has water rights in the North Branch of the Contoocook River and has built a dam at a point on the river approximately 1/4 mile westerly of Hillsboro Lower Village with a penstock extending from the dam to the surge tank located downstream at the confluence of the North Branch and the Contoocook River. The penstock passes through the reservoir area and the site of the dam and diversion channel. The consideration for the flowage easement will be the cost of replacing the existing penstock with a steel lined concrete penstock, the cost of which is included in the estimated cost of the dam and appurtenant works. There are no other utilities affected within the reservoir area.

b. Roads.- The existing first class highway (State Highway No. 9), which connects the Town of Hillsboro and Hillsboro Lower Village, passes through the proposed embankment and diversion canal sites. It is proposed to raise the road approaches and construct a new steel girder highway bridge across the diversion canal which would permit uninterrupted traffic on Route 9 during periods of high water. Construction of the bridge and approaches would proceed with construction of the dam, and highway traffic could be maintained during the construction period by means of a detour as indicated on Plate IV-11. The present stone double-arch highway bridge spanning the North Branch on the Antrim-Hillsboro Lower Village road is inadequate in area for passing flood stage flows and the present approaches would be subject to inundation during periods of high water. A new highway bridge, adequate in design to pass a flood of the 1936 flood magnitude, is proposed to replace the present double arched bridge with the deck and adjacent approaches raised above spillway crest elevation. Access to the Bible Hill Cemetery, which is located approximately 1,500 feet upstream from the east abutment of the dam, is gained from either the Beards Brook Road or Bible Hill Road at the present time. The access passageway to the cemetery via Beards Brook Road would be blocked by the dam and in order to provide access to the cemetery it is proposed to improve the existing roadway from Bible Hill Road. The secondary roads to be abandoned within the reservoir area are the stretch of road between Pine Hill Cemetery and the Bible Hill Road and those sections of Beards Brook and Bible Hill Roads that would be inundated during high water. These roads would, however, be open to traffic during normal times.

The Chief of Engineers of the Highway Department, State of New Hampshire, has been consulted and has agreed that the relocations proposed are a reasonable solution for raising and relocating the network of roads affected by construction of the proposed dam.

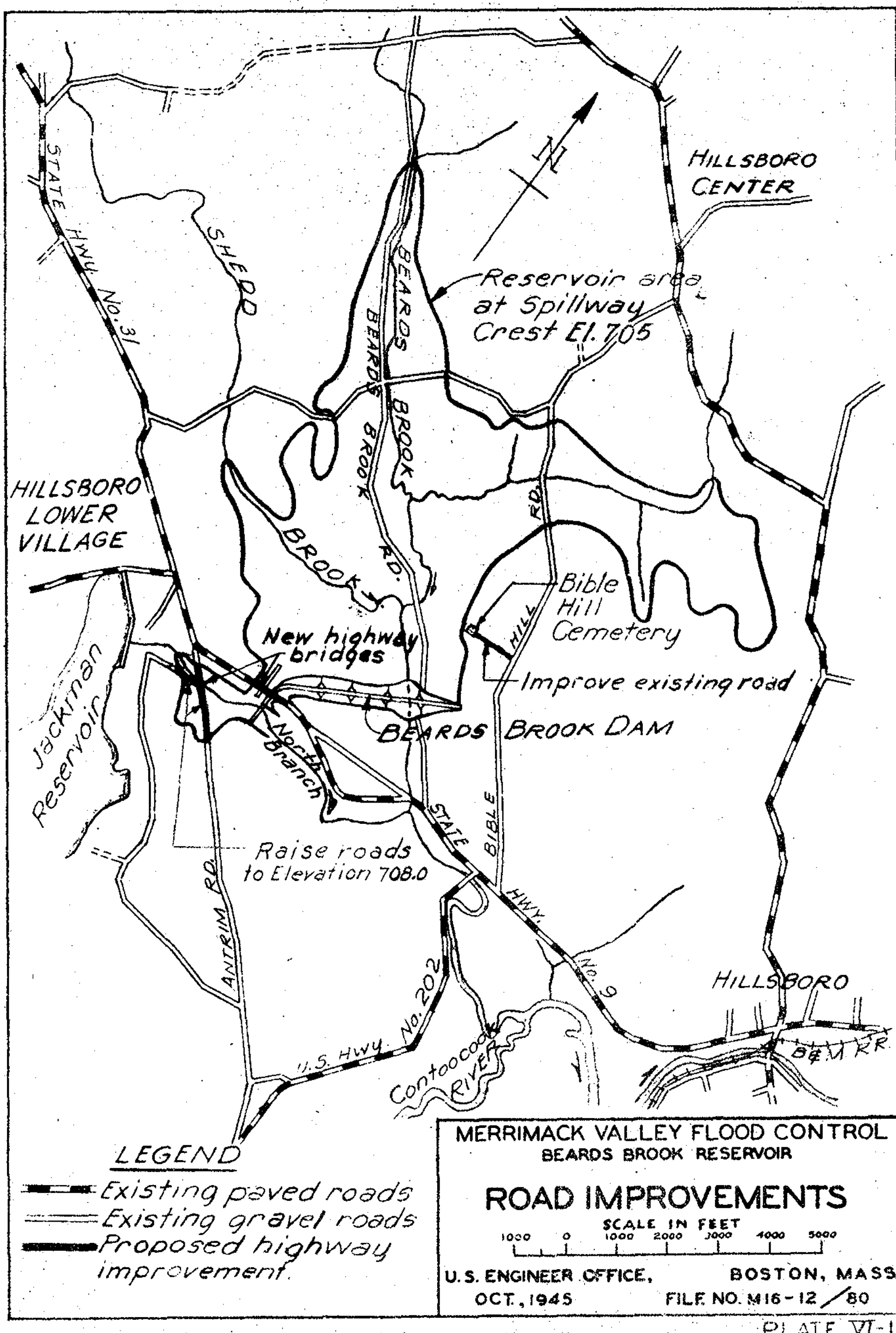
c. Utilities.- The wood stave pipe line (penstock) which supplies water to the surge tank of the existing hydroelectric installation at the confluence of Beards Brook and North Branch, crosses the site of the proposed diversion canal and earth embankment and is owned by the Public Service Corporation of New Hampshire. It is proposed to install the new section of pipe line below the diversion canal and under the dam from the dike on the west bank of the diversion channel to a point approximately 200 feet downstream from the toe of the embankment. The new section of pipe line (penstock) will be a steel lined concrete conduit, sections of which are shown on Plate IV-8.

An existing water main which serves the town of Hillsboro is located in the easterly area of the reservoir, approximately 6 feet under the present ground surface. Relocation is not required within the reservoir area.

d. Power and Telephone Lines.- The existing power and telephone lines paralleling State Highway No. 9 cross through the proposed diversion canal and embankment and will be raised upon completion of road construction work described in paragraph b. The telephone line, bordering that portion of the Antrim-Hillsboro Lower Village Road which requires raising, is to be relocated at the higher road elevation.

e. Method of Accomplishing Work.- It is anticipated that the relocation and raising of roads, power and telephone lines will be accomplished by the respective owners who will be reimbursed by the Government for such work with the exception of the relocated pipe line. The cost of constructing the relocated pipe line, which is owned by the Public Service Company of New Hampshire, is included in the estimated construction cost of the dam, and will be performed by the contractor for the dam and appurtenant works. State Highway No. 9, including the highway bridge, is owned by the State of New Hampshire and all other affected roads and bridges are owned by the Town of Hillsboro; the power lines by the Public Service Company of New Hampshire, and the telephone lines by the New England Telephone & Telegraph Company.

f. Source of Information.- In accordance with the requirements of Circular Letter No. 3570, Real Estate No. 62, dated 21 February 1945, subject: "Real Estate Functions of Division Offices", the method of disposing of utilities lying in the reservoir area was developed in collaboration with representatives of the Real Estate Division of the Office of the Division Engineer, New England Division. The general description of the real estate involved and data as to its acquisition cost were taken from the "Report-Real Estate Cost, Beards Brook Reservoir, N.H.", dated 20 October 1944, prepared by the Division Engineer, New England Division.



HILLSBORO CENTER
STATE HWY. No. 31
SHEDD
BEARDS BROOK
BEARDS BROOK RD.
Reservoir area at Spillway Crest El. 705
HILLSBORO LOWER VILLAGE
BROOK
RD.
Bible Hill Cemetery
New highway bridges
Improve existing road
BEARDS BROOK DAM
Jackman Reservoir
North Branch
HILL
BIBLE
STATE HWY. No. 9
Raise roads to Elevation 708.0
ANTRIM RD.
U.S. HWY. No. 202
Contoocook RIVER
HILLSBORO
B & M RR
LEGEND
Existing paved roads
Existing gravel roads
Proposed highway improvement
MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR
ROAD IMPROVEMENTS
SCALE IN FEET
1000 0 1000 2000 3000 4000 5000
U.S. ENGINEER OFFICE, BOSTON, MASS.
OCT., 1945 FILE NO. M16-12/80

PLATE VI-1

APPENDIX VII

RECREATIONAL FACILITIES

To accompany definite project report
dated November 1945

DEFINITE PROJECT REPORT

BEARDS BROOK RESERVOIR

APPENDIX VII - RECREATIONAL FACILITIES

CONTENTS

PLATES

DEFINITE PROJECT REPORT
BEARDS BROOK RESERVOIR

APPENDIX VII

RECREATIONAL FACILITIES

a. Introduction.- In accordance with the provisions of Circular Letter No. 3579, dated 26 February 1945, subject: "Recreation Facilities at Reservoirs", the Directors of the several state agencies responsible for the promotion of public recreational facilities in the State of New Hampshire were consulted regarding their possible interest in the development of recreational facilities in the Beards Brook Reservoir area. The agencies consulted were the State Planning and Development Commission and the Forestry and Recreation Department. The latter agency exhibited a definite interest in the development of a recreational lake in accordance with the plans shown on Plate VII-1. This lake has a fixed weir at elevation 640, extends about 1 mile upstream from the dam and has a maximum depth of 15 to 20 feet. The Director of the Forestry and Recreation Department visualized an extensive use of a lake of this type for swimming, boating, fishing, camping and general recreation purposes. The Director of the State Planning and Development Commission expressed a similar interest in the development of the reservoir area with the suggestion that the lake level be maintained at elevation 650 to permit the shores of the lake to extend to the steeper side slopes of the reservoir basin. This latter agency visualized the development of summer cottage sites on the bluffs surrounding the rim of the reservoir. The capacity of the reservoir at elevation 640 is 1,000 acre-feet and at elevation 650, 3,400 acre-feet or about 10% of the total reservoir capacity. In either case, a fixed weir control is considered essential to avoid reliance on manual operation of the gates, especially in such instances as this where the time interval between sequences of operation on a rising flood would be very short - a matter of a few hours.

b. Estimated Cost.- The estimated cost of providing a recreational lake in the reservoir is summarized as follows:

Lake Level at El. 640:

Fixed Weir	$ 2,600.
Clearing 170 acres	27,400.
Access Roads and Parking Areas	10,000.
Sub-Total	$40,000.
Contingencies and Government Costs, 25%.	10,000.
TOTAL	$50,000.

Lake Level at El. 650:

Fixed Weir	$ 3,800.
Clearing 210 acres	34,200.
Access Roads and Parking Areas	10,000.
Sub-total	$48,000.
Contingencies and Government Costs,25%.	12,000.
TOTAL	$60,000.

Provision for such expenditures as outlined above have not been included in the estimated cost of the project summarized in paragraph 14 of the text of the report.

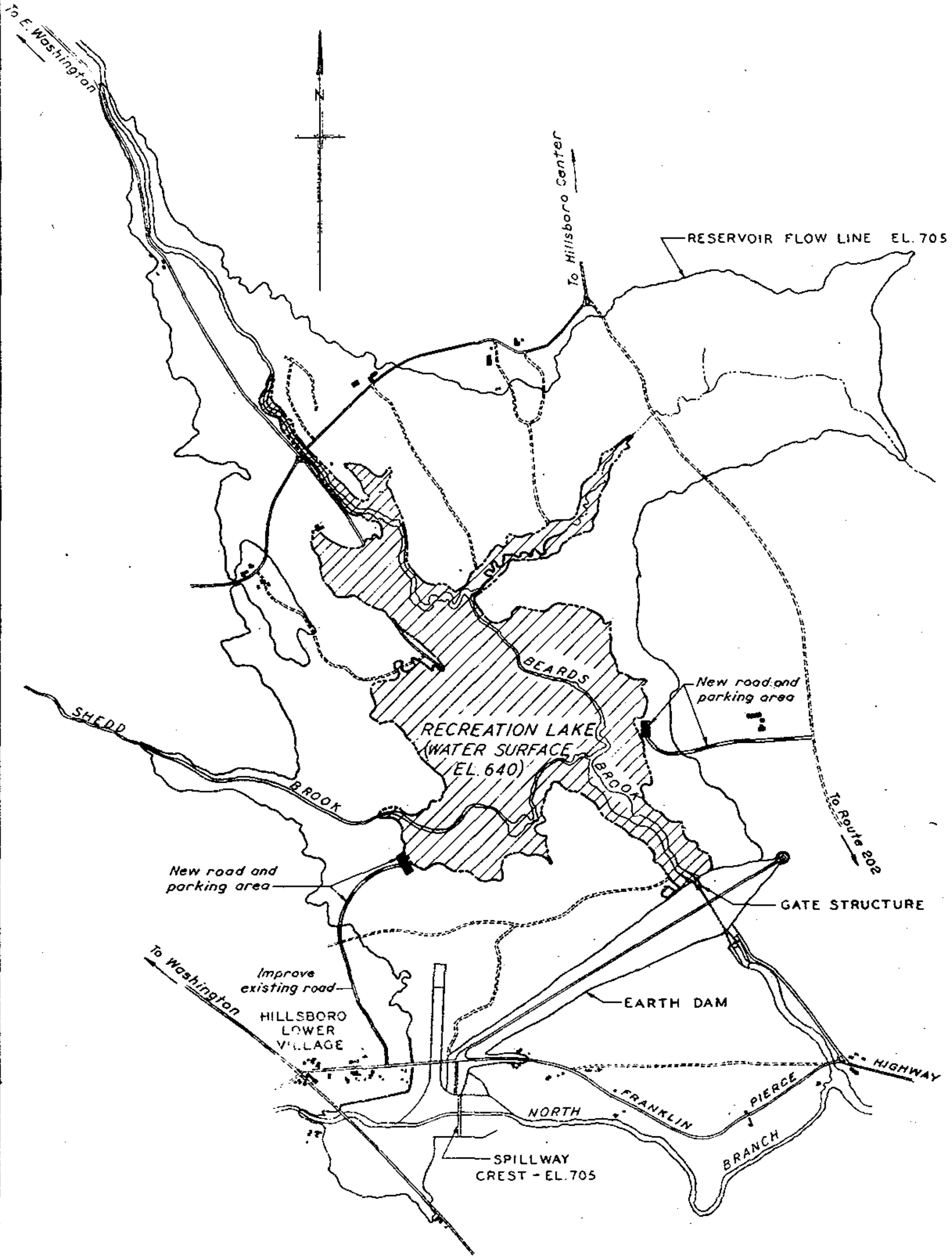
c. Benefits.- It appears that the Federal Government would obtain little if any financial return for the expenditure of recreation facility funds. Arrangements whereby the State would enter on Federal lands and establish auxiliary recreational facilities have not been discussed in detail, but such leases or rights of entry would undoubtedly be for nominal fees. This is in line with the policy set forth in the Flood Control Act of December 1944, which, as quoted in Circular Letter 3579, states: "That preference shall be given to Federal, State or local governmental agencies and licenses may be granted without monetary consideration, to such agencies for the use of areas suitable for public park and recreational purposes, when the Secretary of War determines such action to be in the public interest........".

A recreational lake development of this character, however, would result in a reduced maintenance cost of the reservoir area by virtue of eliminating the periodic clearing of trees in the low levels of the reservoir, which possibly would be killed by occasional flooding. In addition, it is believed that it will aid in promoting the general sanitation and sightliness of the reservoir area.

d. Operation of Reservoir.- The expected fluctuation of a low level lake during the recreation season of June through October has been investigated. Had a lake with weir crest at elevation 640 been in existence in the Beards Brook area for the past 20 years, the level of the lake would not have risen more than approximately 3 feet during the recreation season except in the instance of the flood of September 1938 and once again in July of 1944. The design of the dam or the regulation of the reservoir is not affected by the creation of a low level lake except for the provision of the fixed weir upstream of the center gate. If a lake to elevation 650 were provided, it would be necessary to revise the proposed method of gate operation to compensate for the approximately 10% loss in flood control storage.

e. Clearance with Other Agencies.- Contacts have been made also with the Fish and Wildlife Service of the Department of the Interior and the Fish and Game Department of the State of New Hampshire regarding the interest of these agencies in the reservoir and to obtain suggestions on possible features to be incorporated in the dam for the promotion of fish life. Definite suggestions have not been received from the Department of the Interior. The State Department, however, has expressed itself as opposed to the creation of a permanent lake on the grounds that Beards Brook is one of the best trout streams in the region. This agency has stated that the warming of the water resulting from creation of the lake and the prohibition of fish migration back and forth through the dam is inimical to the present fish life of the brook. It has requested further that baffles be installed in the outlet conduit to provide resting places at intervals for the easy upstream advance of young fish. These baffles, of course, would have to be of such construction as to wash out during flood flows. Time has not permitted the conflict in interests between the several State agencies to be resolved.

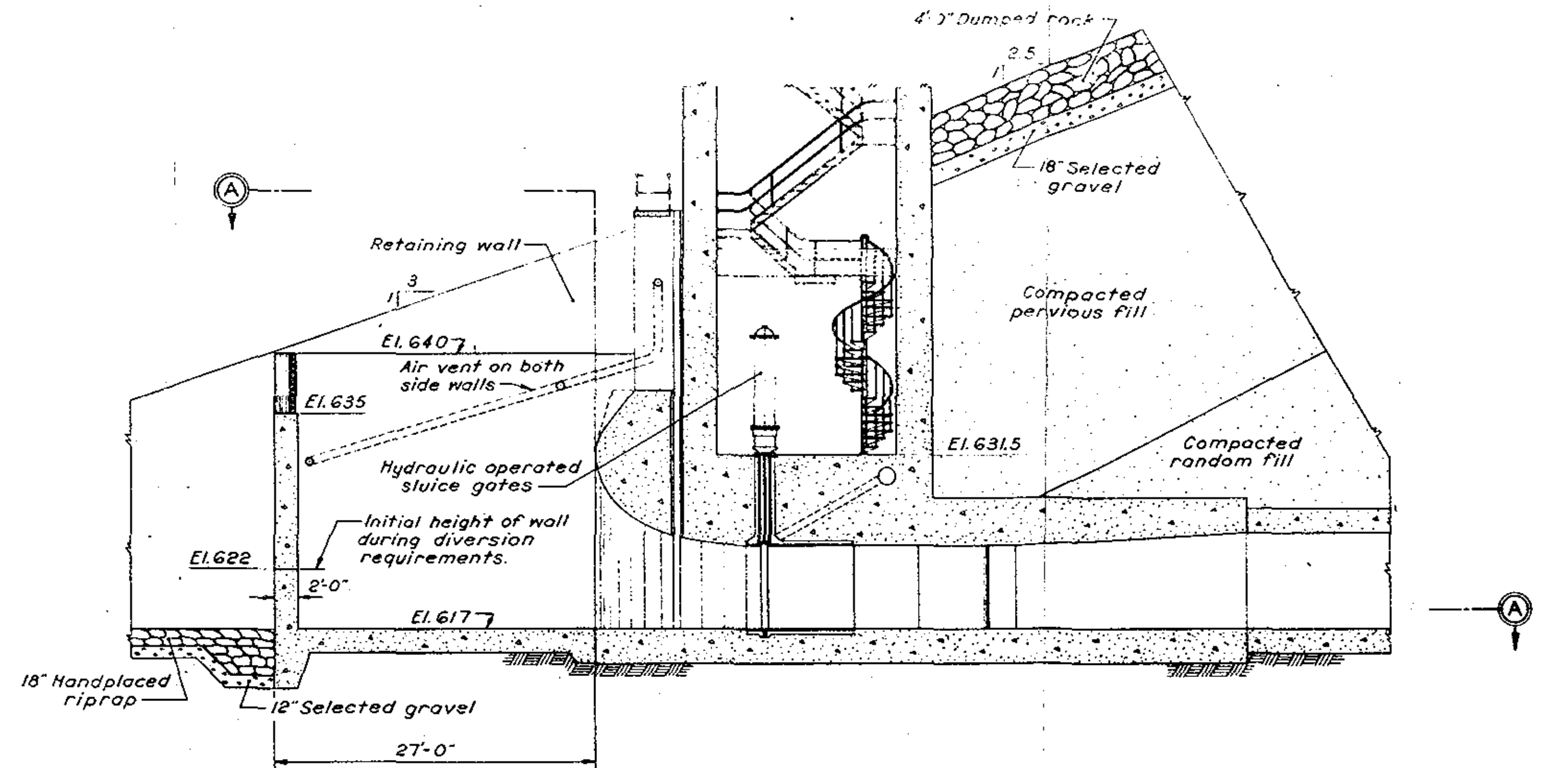
f. Recommendation.- It is recommended that the studies of the several State agencies and a coordination of their interests be allowed to progress in normal course and that this report be amended at a later date to include the final agreement of these agencies on the measures to be incorporated for recreation and wildlife.



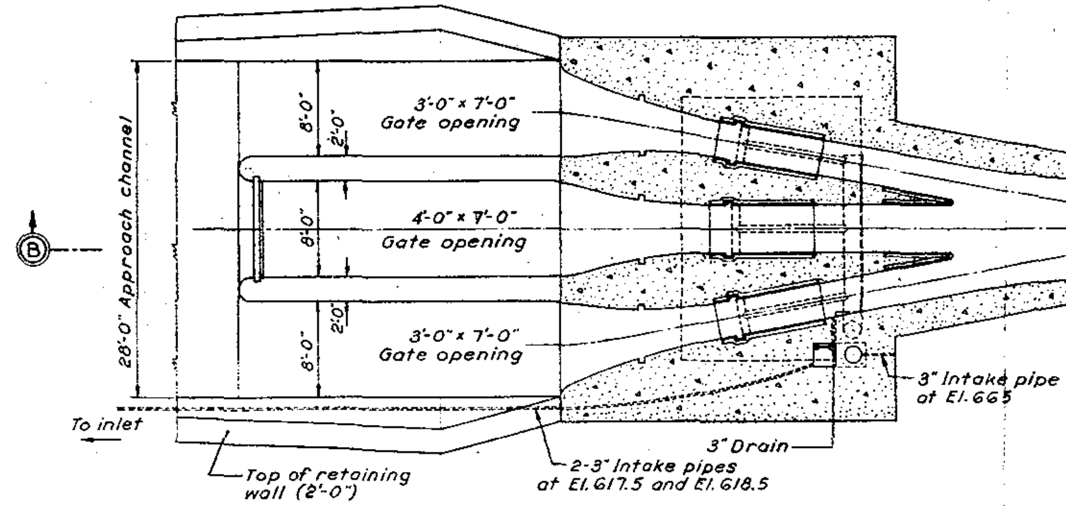
PLAN

800' 0 800' 1600'

SCALE: 1 INCH = 800 FEET



SECTION B-B



SECTION A-A

Note:- Elevations are based on U.S.C. & G.S. datum, Mean Sea Level.

12'0 5' 10' 15' 20' 25'

SCALE: 1/8 INCH = 1 FOOT

(UNLESS OTHERWISE NOTED)

THIS DRAWING REDUCED TO ONE-HALF THE ORIGINAL SCALE.

NO.	CHARACTER	DATE	APPR'D
	REVISIONS		

MERRIMACK VALLEY FLOOD CONTROL
BEARDS BROOK RESERVOIR

PLAN OF RECREATION LAKE & GATE STRUCTURE WITH WEIR

SCALE AS SHOWN

U. S. ENGINEER OFFICE, BOSTON, MASS. OCT. 1945

APPROVAL RECOMMENDED: John E. Allen, CHIEF ENGINEERING DIVISION

APPROVED: COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER

SUBMITTED: A. B. Brown, CHIEF DESIGN BRANCH, SR. ENGINEER

ENGINEER

FILE NO. MIS-12/81